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Tests of Reinforced Concrete Tee Beams

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TESTS OF REINFORCED CONCRETE TEE BEAMS

BY

FLOYD SINNOCK HEWES
AND
CLARENCE AVISE HEWES

THESIS

FOR

DEGREE OF BACHELOR OF SCIENCE

IN

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May 29, 1906

This is to certify that the following thesis prepared under the direction of Professor A. N. Talbot, Head of the Department of Theoretical and Applied Mechanics, by

CLARENCE AVISE HEWES and FLOYD SINNOCK HEWES

entitled TESTS OF REINFORCED CONCRETE TEE-BEAMS

is hereby approved by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

Ira O Baker

Head of Department of Civil Engineering

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I Introduction

One of the most interesting features of modern structural engineering is the development of the use of concrete reinforced with steel. The use of this structural material has had a marvelous growth in the last few years and now it has replaced to a great extent the use of timber, masonry, and steel. When reinforced concrete first came into use its properties were shrouded in mystery, but investigations that have been and are now being carried on have served to give us a good deal of information on the behavior of this new material under stress. These investigations have been carried on not only by private parties and technical schools but also by the United States Government. Lately the United States Geological Survey has taken up the study and to secure some uniformity in tests made in different laboratories throughout the country, they have taken charge of the supply of the materials. The materials for these tests were furnished under their direction by the American Society of Civil Engineers, the American Society for Testing Materials, the Association of American Portland Cement Manufacturers, and the Amer-

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ican Railway Engineering and Maintenance of Way Association.

Reinforced concrete has been used for a number of years in the floors of buildings. In those cities that have any articles in their building ordinances covering such floor construction, it has always been an open question as to how much of the flange was effective in determining the area of the steel reinforcement in the web. There have been a few tests made on reinforced concrete tee-beams in late years in different parts of the country to throw some light on this question but none of these were carried out along scientific lines, the beams being generally tested by loading them with pig iron. According to the knowledge of the authors, the investigations on tee-beams included in this thesis are the first ones to be carried out along scientific lines.

II Description of Materials

The materials for these tests were furnished by the Joint Committee as described on page 1, with the exception of the steel which was furnished through the courtesy of the manufacturers.

The stone was a good quality of Kankakee limestone, ordered to pass a 1-in. screen and to be retained on a $\frac{1}{4}$-in. screen. It contained about 50 per cent of voids.

The sand was of good quality and fairly clean. It contained 28 per cent of voids. The table below gives the result of a mechanical analysis of this sand.

ANALYSIS of SAND.

Sieve No.	Per cent Passing
4.	100
10	73
20	36
30	12
74	5
100	2

The cement used was a mixture of several brands of Portland cement of recognized quality. The following table gives the tensile strength of both neat cement and mortar mixed in the

proportion of one part cement to three of sand by weight. The briquettes were thumb-rammed, of standard section, and were stored in damp air for one day and under water the remainder of the time.

TENSILE STRENGTH of CEMENT

Ref. No.	Ultimate Strength in lb. per sq in.			
	Age 7 days		Age 60 days	
	Neat	1:3 Mortar	Neat	1:3 Mortar
Average				

The concrete was mixed in the proportion of 1:2:4, thus securing a very rich mixture. All materials were measured by loose volume in buckets. The mixing was done by hand with shovels. The stone was wetted a day or so before being used and then used while still wet. The sand and cement were first mixed dry; the stone was added and turned several times, after which the water was added and the mass mixed

until uniform in appearance. A very wet concrete was used. The table below gives the result of tests on 6-in. cubes. The average ultimate strength for cylinders 16 in. high by 8 in. in diameter was 1600 lb. per sq. in.

RESULT of TEST of 6 IN. CUBES

Concrete as in Beam No.	Age at test. days	Max. Applied Load in lb.	Comp. Str. lb. per sq. in	Av. Res. lb. per sq. in
1	59	62 120	1 720	1970
"	"	84 690	2 350	
"	"	66 390	1 840	
3	57	42 120	1 170	1280
"	"	49 620	1 380	
"	"	46 200	1 280	
5	58	69 800	1 940	2100
"	"	81 500	2 260	
7	56	57 600	1 600	1830
"	"	71 100	1 980	
"	"	68 300	1 900	
8	54	68 100	1 890	2020
"	"	55 600	1 540	
"	"	94 500	2 620	
		Total Av = 1840		

The horizontal reinforcement consisted of $\frac{3}{4}$-in. bars. Part of these were Johnson Corrugated bars with a net section of .56 sq. in. The remainder were Plain round bars. Results of tensile tests of these bars are given below, the steel being picked at random from the different beams. The steel for the stirrups was $\frac{1}{2}$-in. Johnson Corrugated bars.

TENSILE STRENGTH OF STEEL

Steel as in Beam No.	Elastic Lim. Pounds	Ultimate St. Pounds	Per cent Elongation	Elastic Lim. lb. per sq. in.	Ultimate St. lb. per sq. in
		Plain $\frac{3}{4}$-in Round Bars.			
4	17 600	26 000	33	39 900	58 800
"	16 200	24 700	33	36 700	55 800
"	16 600	24 500	35	37 600	55 400
"	16 000	23 100	37	36 200	52 300
9	17 400	24 800	35	39 400	56 100
"	17 400	26 000	30	39 400	58 800
"	17 100	26 500	33	38 700	60 000
"	17400	25 700	31	39 400	58 100
"	17 200	25 400	33	38 900	57 500
"	17 700	28 500	29	40 000	64 500
"	16 800	26 000	32	38 000	58 800
		Average	33	35 800	57 800
		$\frac{3}{4}$-in. Johnson Bars			
2	29 700	48 500	15	52 800	86 200
"	30 800			54 700	
3		49 300	13		87 700
"	29 000	46 200	14	51 500	82 300
"	30 000	53 500	14	53 300	95 000
"	30 000	51 100	17	53 300	91 000
5	29 000	47 710	15	51 500	84 900
"	31 000	52 200	15	55 000	92 900
"	31 300	52 100	12	55 600	92 700
"		39 700	22		70 400
"	28 000	47 400	12	49 800	84 400
"	31 000	54 400	14	55 000	96 700
		Average	15	53 200	87 600

III. Description of Specimens.

The test included nine beams, 11 ft. long with a clear span of 10 ft.. These were tested on the 600 000 lb. Riehle testing machine in the Laboratory of Applied Mechanics of the University of Illinois. The following dimensions were the same for all the beams: thickness of flange $3\frac{1}{4}$ in., width of web 8 in., depth over all 12 in., and depth of beam from top fiber to center of gravity of the steel 10 in.. The percentage of reinforcement was obtained by dividing the area of the steel rods by the product of the width of the flange and the effective depth of the beam. It was intended to have this as nearly 1 per cent as possible and it varied from .92 to 1.10 per cent. The above figures on the size and reinforcement were made as nearly constant as possible to eliminate variables. There were three different widths of beams — 16, 24, and 32 in. — and three of each width.

The rods were placed symmetrically so that the effect of each rod extended over an equal area. In some of the beams, as indicated in the diagrams on page 14, part of the rods were turned up at the ends, so that the end of these came within 4 in. of the top of the

beam. Ten stirrups, made of $\frac{1}{2}$-in. Johnson bars, were used in each beam and placed as shown. In a few of the beams, several $\frac{1}{2}$-in. Johnson old style bars were imbedded in the flange just below the surface, transversely to the span. The bent up bars and the stirrups were intended to assist in resisting the shearing stresses, and the bars imbedded in the flange were intended to reinforce the edges of that part of the beam. Most of the beams had hooks of $\frac{1}{4}$-in. Johnson bars imbedded in the concrete beyond the clear span to assist in handling them.

The beams were made on the concrete floor of the Laboratory. Strips of building paper were laid over the floor to prevent the concrete from adhering to it. The forms consisted of 2-in. plank for the sides and ends held together by clamps made of 2×4-in. verticals, a cross bolt, and a 2×4-in. horizontal strut. See pg. 12. To prevent the planks from warping and from absorbing water from the concrete along the face of the beams, they were soaked in a tank for several weeks before being used and then used when still thoroughly soaked.

All concrete was mixed by hand and proportioned by loose volume. Generally only one

batch was used for a beam and the concrete was of such consistency that splashing occured with heavy tamping. After a form was set up, a layer of concrete 1 to $1\frac{1}{2}$ in. thick was placed in the bottom and a level surface was made on this by means of a wooden template. The reinforcement was then placed in the desired position, special attention being paid to placing the concrete around the steel bars and stirrups. This was done by means of a trowel and wooden rammers. The rest of the concrete was put in in layers of about 3 in., spaded with a trowel along the sides and ends, tamped with an iron tamper, spaded again and then retamped.

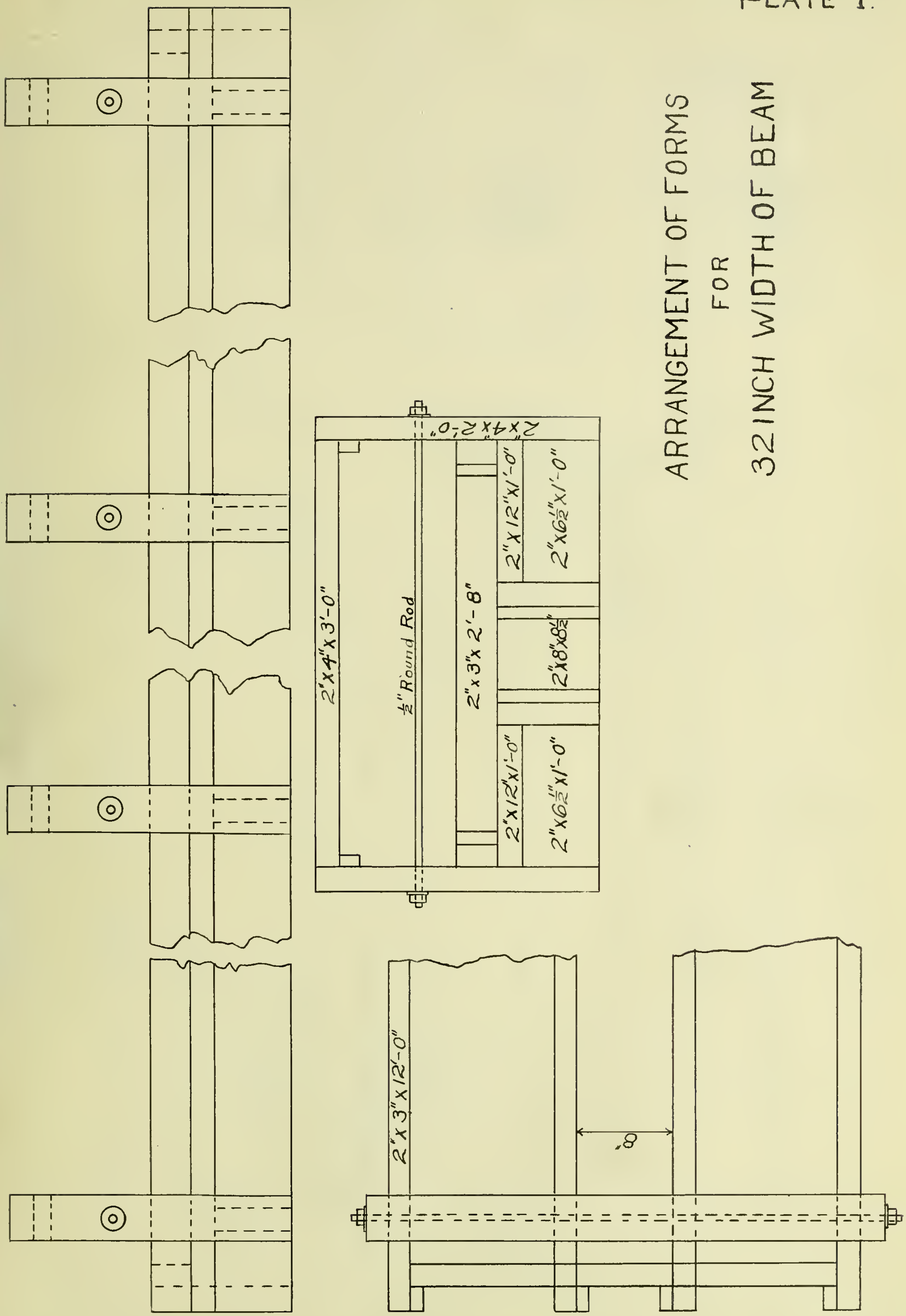
In this way the forms were filled. As there was a form for each beam, they were left on until a day or so before testing. Upon their removal it was seen that the above method of filling the forms gave a very good appearance to the faces of the beam. For the sake of appearance the top of the beam was finished with a thin coat of 1 to 2 mortar. The beams were sprinkled twice daily from the time of making to within five or six days before testing. A set of three 6-in. cubes and one cylinder 16 in. high and 8 in. in diameter were planned for

each beam. The concrete for all the test pieces of this thesis was made by an experienced concrete laborer assisted by the authors.

DATA on BEAMS

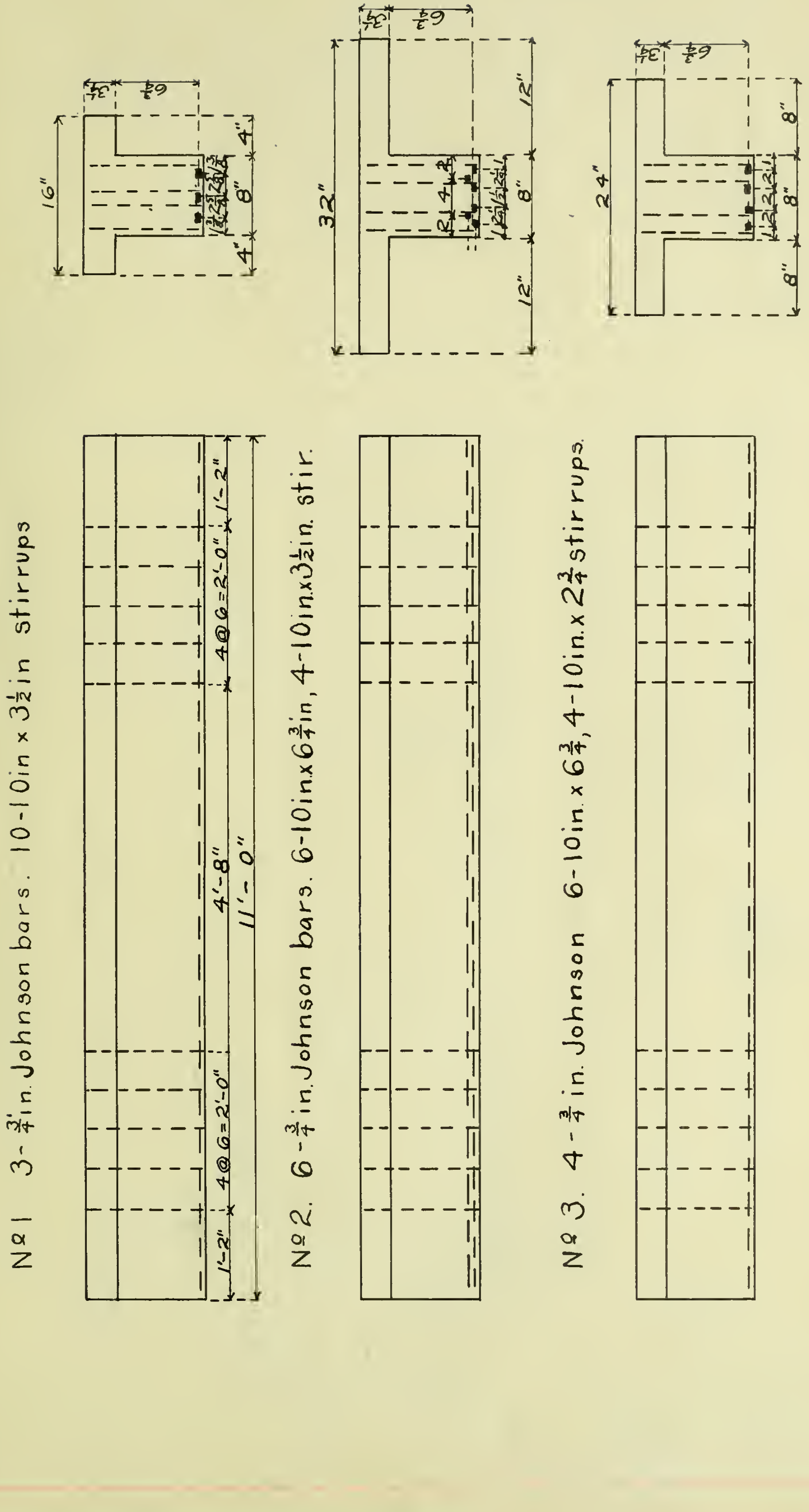
Beam No.	Width of Flange inches.	Amount and Kind of Reinforcement.	Per cent of Metal	No. of Bars turned up	Age at Test days
1	16	3 - $\frac{3}{4}$ in. Johnson	1.06	0	58
4	"	4 - $\frac{3}{4}$ " Plain R'd	1.10	0	57
7	"	4 - $\frac{3}{4}$ " " "	"	0	60
3	24	4 - $\frac{3}{4}$ " Johnson	0.94	0	64
6	"	5 - $\frac{3}{4}$ " Plain R'd	0.92	2	60
8	"	5 - $\frac{3}{4}$ " " "	"	"	58
2	32	6 - $\frac{3}{4}$ " Johnson	1.06	0	58
5	"	6 - $\frac{3}{4}$ " "	"	2	62
9	"	7 - $\frac{3}{4}$ " Plain R'd	0.97	3	59

PLATE I.



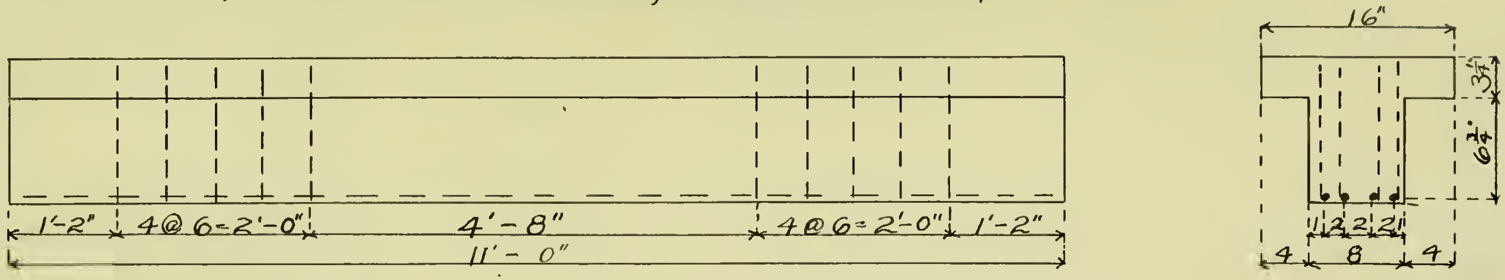
ARRANGEMENT OF FORMS
FOR
32 INCH WIDTH OF BEAM

PLATE II.

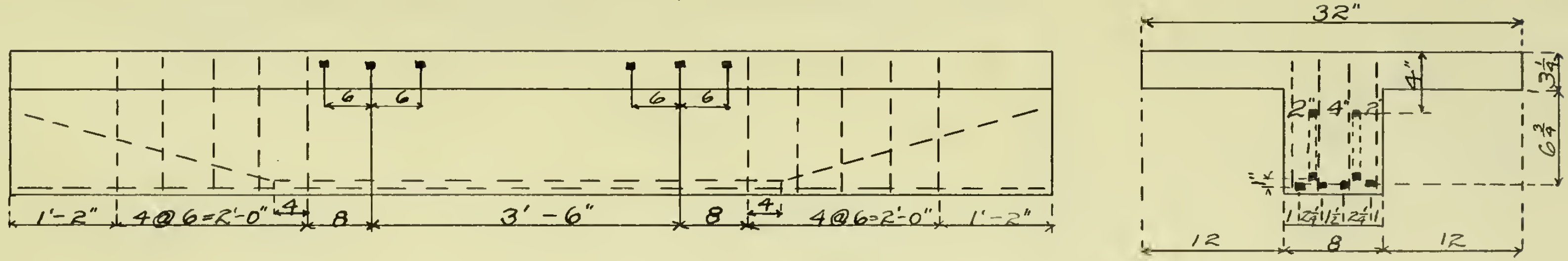


No 1   3-3/4 in. Johnson bars. 10-10 in x 3 1/2 in stirrups
No 2. 6-3/4 in. Johnson bars. 6-10 in. x 6 3/4 in, 4-10 in. x 3 1/2 in. stir.
No 3. 4-3/4 in. Johnson 6-10 in. x 6 3/4, 4-10 in. x 2 3/4 stirrups.
16"
4"
8"
32"
12"
24"
4@6=2'-0"
1'-2"
4'-8"
11'-0"

No 4. 4-$\frac{3}{4}$ in. Round bars. 6-10"x$6\frac{3}{4}$", 4-10"x$2\frac{3}{4}$" stirrups.



No 5. 6-$\frac{3}{4}$ in. Johnson bars. 6-10"x$6\frac{3}{4}$", 4-10"x$2\frac{1}{4}$" stirrups.



No 6. 5-$\frac{3}{4}$ in. Round bars. 8-10"x$3\frac{1}{2}$, 2-10"x$6\frac{3}{4}$" stirrups

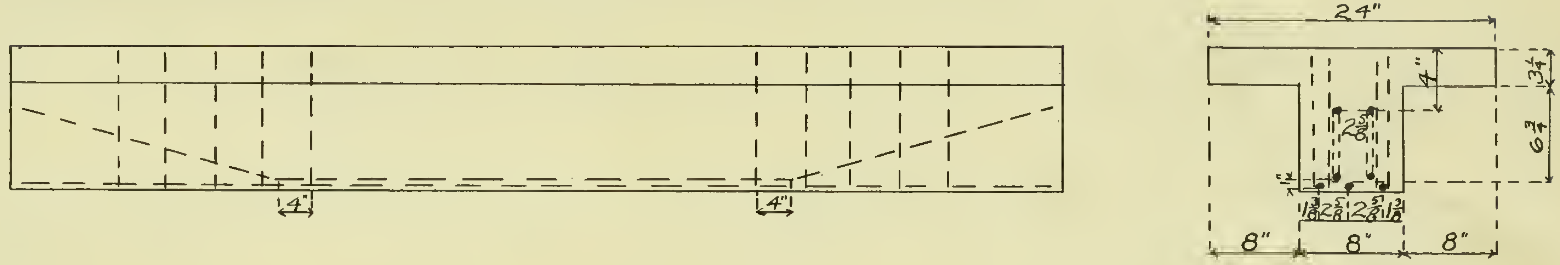
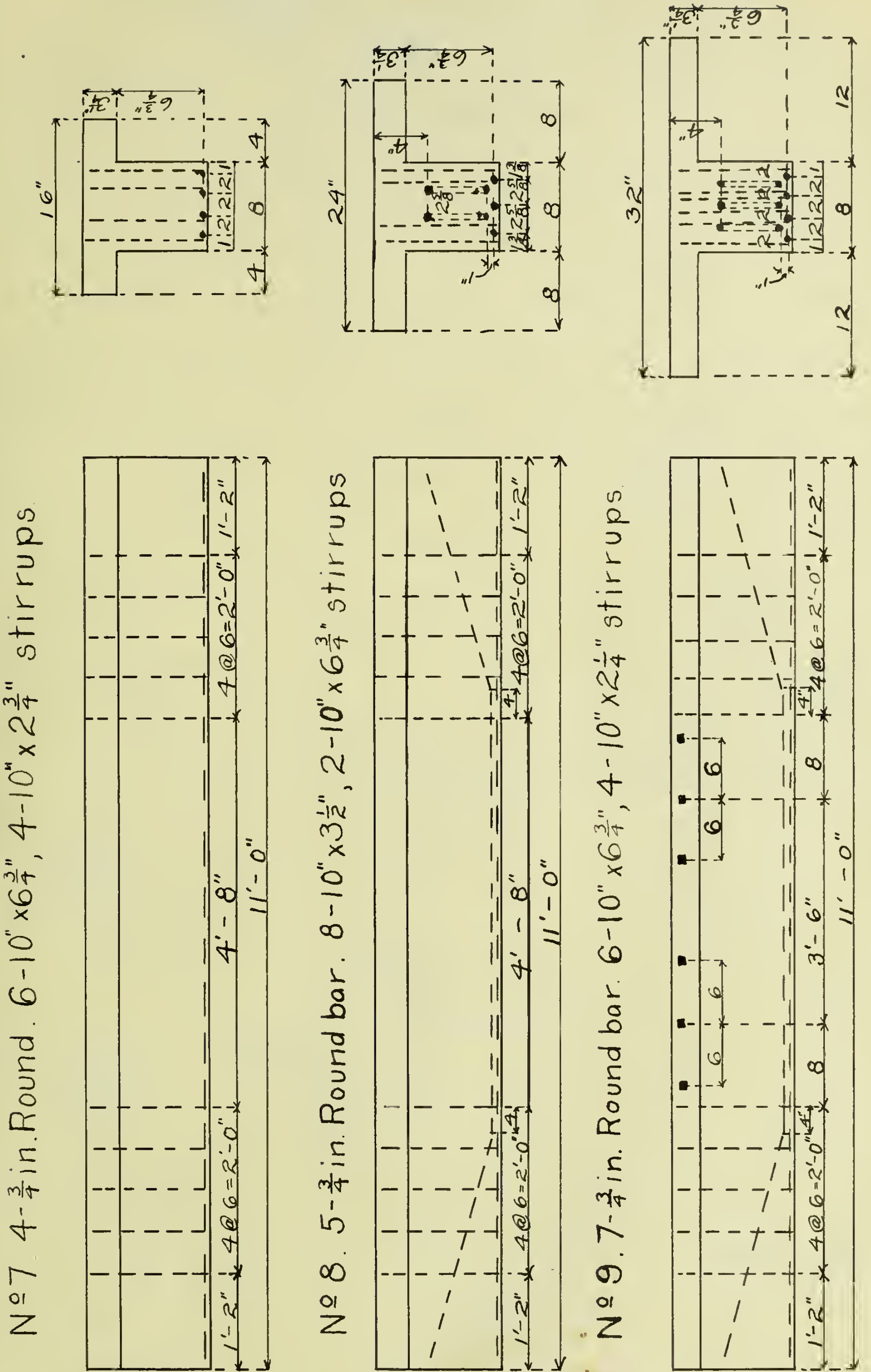


PLATE IV.


No 7. 4-$\frac{3}{4}$ in. Round. 6-10" x 6$\frac{3}{4}$", 4-10" x 2$\frac{3}{4}$" stirrups
No 8. 5-$\frac{3}{4}$ in. Round bar. 8-10" x 3$\frac{1}{2}$", 2-10" x 6$\frac{3}{4}$" stirrups
No 9. 7-$\frac{3}{4}$ in. Round bar. 6-10" x 6$\frac{3}{4}$", 4-10" x 2$\frac{1}{4}$" stirrups
16"
24"
32"
4'-8"
3'-6"
11'-0"
1'-2"
4@6=2'-0"

IV. Description of Tests.

On account of the great weight of the specimens, about 1400 to 2000 lb., and of their shape, trouble was anticipated in moving them; but by means of a portable trestle crane provided with two movable blocks on an overhead trackway, they were moved to and from the testing machine without much difficulty. The hooks at the ends of the beams provided a good hold for the hooks of the pulley.

All the beams were tested on the 600 000-lb. Riehle testing machine and the cubes, cylinders, and steel on the 100 000-lb. Riehle machine. The movement of the head of the machine for testing beams was $\frac{1}{20}$ in. per minute and for the cubes, cylinders, and steel $\frac{1}{10}$ in. per minute. The load was applied at the one-third points by means of one or two (according to the beam) 10 in., 25 lb. I-beams, $5\frac{1}{2}$ ft. long. To distribute the load across the beam, two 2-in cast iron plates faced on both sides, 3 in. wide and 2 ft. 8 in. long were used. On these rested $4\frac{1}{2}$-in. or 2-in. turned steel rollers, (according to the beam) upon which the I-beams were placed. On top of the I-beams there was placed a $\frac{3}{4}$-in. plate, 3 in. wide, and 10 in. long. The supports had

curved bottoms to permit rocking, and plates 8 in. x $1\frac{1}{8}$ in. x 12 in. were used as bearing plates. These plates together with the bearing plates on top of the beam were imbedded in plaster of paris and allowed to harden under the weight of the beam and the apparatus used in loading before the load was applied. Thus an even bearing was secured and the above arrangement of rollers and pedestals gave very great freedom to longitudinal movement of the beam.

The deflections were obtained at the center by means of a thread stretched between two points over the supports and about 7 in. from the top of the beam. Behind this, at the center of the beam, was fastened a mirror to which a paper scale was pasted. By lining up the thread and its reflection in the mirror, parallax was avoided and readings accurate to .01 in. obtained. In the test of beam number 4, a deflectometer was placed on the bed of the machine beneath the center of the beam and deflections obtained to .001 in.. It would seem from this that the deflectometer method was the more accurate, but in this method any longitudinal movement of the beam during the test or any settling of the beam would affect

the readings of the deflectometer.

The same extensometers that had been used on the rectangular beams in previous tests were used in these tests. The distance between the centers of the top and bottom dials was $20\frac{1}{4}$ in. and the apparatus was so placed that the distance from the center of the top dial to the upper fiber was $4\frac{1}{2}$ in.. In order that the yokes of these extensometers might clear the flange, wooden blocks made of 2-in. plank were fastened to them by screws. These blocks were so arranged that when the apparatus was placed on the beam they would not touch the flange. It was considered more desirable to place both of the contact points on the web on account of the lack of knowledge as to the distribution of the stresses in the flange. On the edge of the blocks nearest to the web, two nails were driven and then filed to a point for bearing points. As there were three different widths of flanges, three different sets of blocks were required. For the same reason, three sets of rods were required to connect the two yokes.

The greatest difficulty with this apparatus was experienced in the buckling of the yokes, thus causing the extensometers and contact points

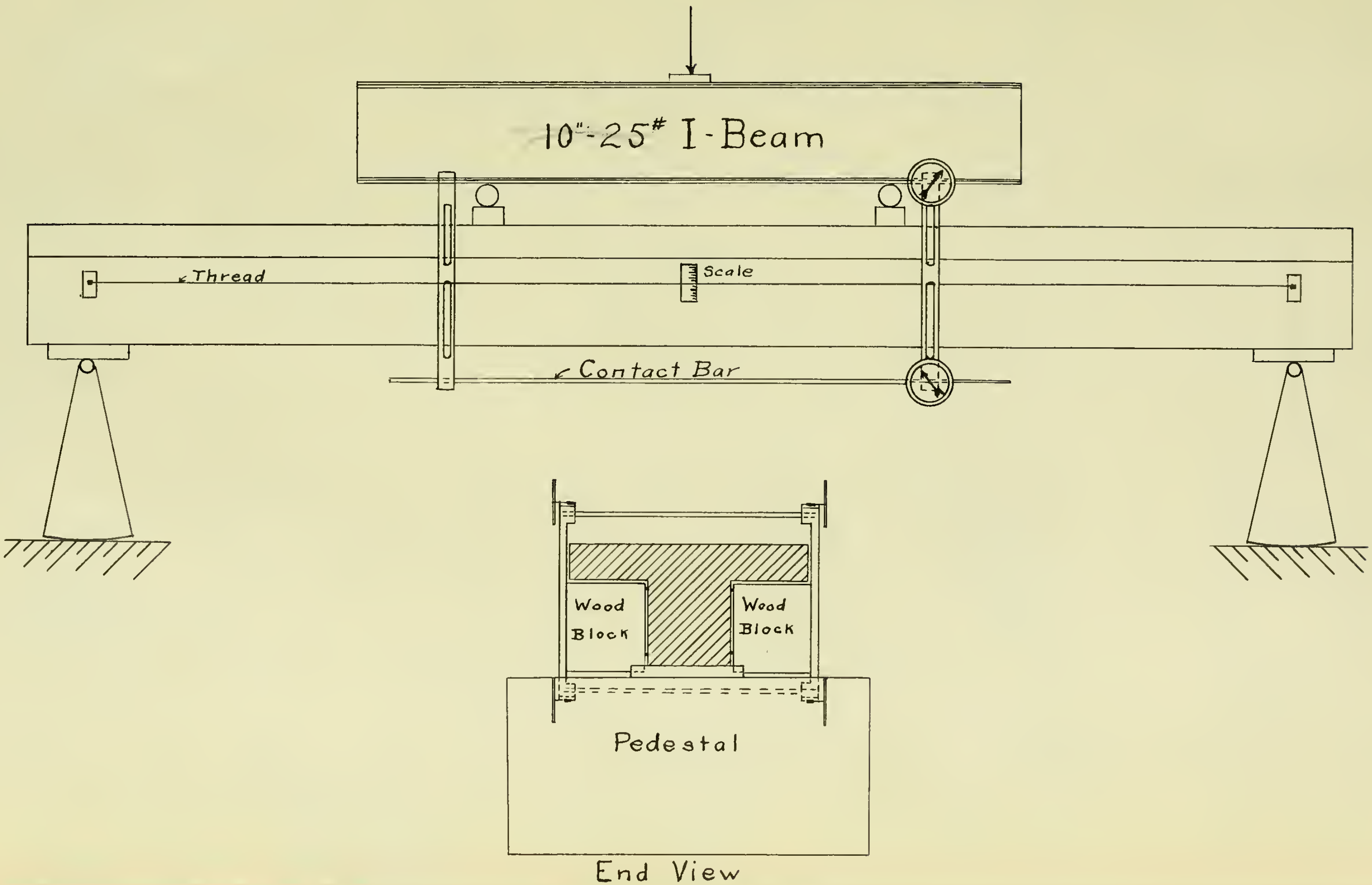
to be thrown out of a plane. The extensometers were tried on all the beams but only four trials were anything like successful. In the other tests the results were too untrustworthy to depend upon on account of the lack of rigidity of the apparatus. In the beams that were tested with the $4\frac{1}{2}$-in. rollers mentioned above, the gage length was 32 in., thus bringing the extensometers inside the load points. None of the attempts with this gage length was successful. All the other tests were made with a gage length of 48 in..

The beams were tested when 60 days old. In some cases the time overran 3 or 4 days, the greatest variation from 60 days being 4 days.

Plate V.

Arrangement of Testing Apparatus

Scale $\frac{3}{4}$ in = 1 ft.

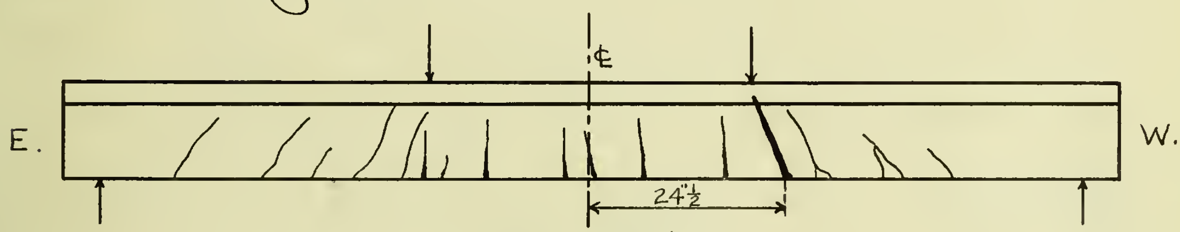


V. Observed Data.

Following is a description of the behavior of each beam during the test and a diagram of each beam showing the cracks and the dimensions of the principal ones.

Beam No. 1. Width of flange 16 in. Weight including testing apparatus 2360 lb. The first cracks to appear were two small tension cracks between the load points. These were first noticed at a load of 16 000 lb. and were about 2 in. high. For the next few thousand pounds about eight more were noticed and at a load of 20 000 lb. the first diagonal one appeared. This was 4 in. outside of the West load point and was 3 in. high. At 26 000 lb. this one was only 2 in. from the flange and extended towards the load point. As the load increased a great number of diagonal cracks developed at the stirrups. At 40 000 lb. the above mentioned diagonal cracks widened and extended higher and the beam just about held the load. After reaching 42 000 lb. the load fell back to 41 000 lb and then slowly rose to 44 500 lb., the maximum. During this period, the tension cracks opened up quite rapidly while the diagonal ones re-

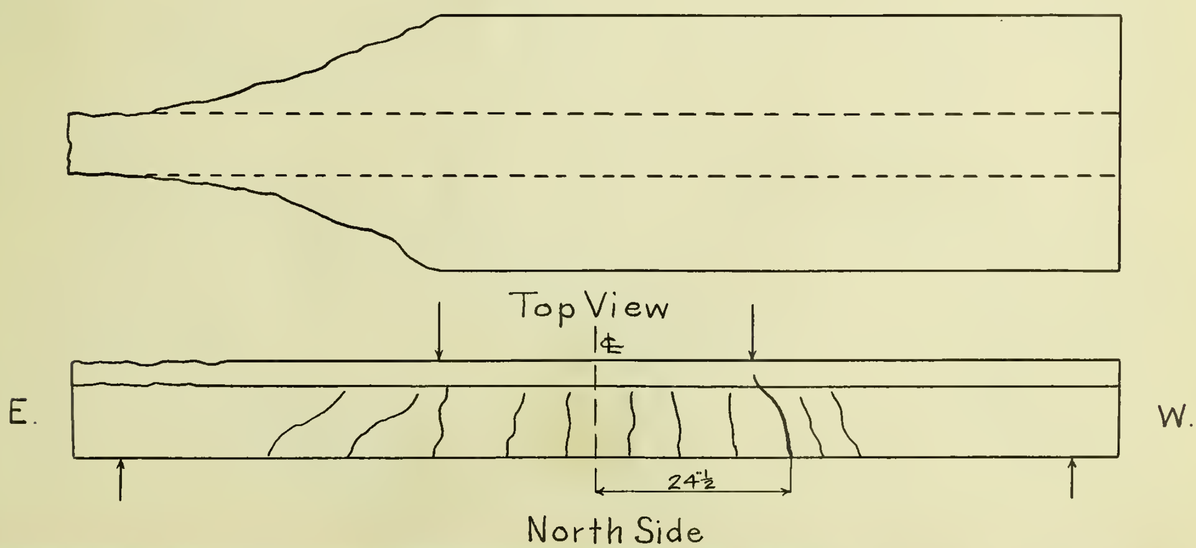
mained about the same. At 44 000 lb. the diagonal crack at $24\frac{1}{2}$ in. West of the center – mentioned above as the first diagonal to appear – was $\frac{1}{32}$ in. wide and larger than any other. The beam held up the load well after the maximum and at 43 800 lb. the concrete crushed on top under the West load point. The deflection at the maximum load was 1.34 in., and at crushing of the concrete 1.60 in. Failure was by tension.



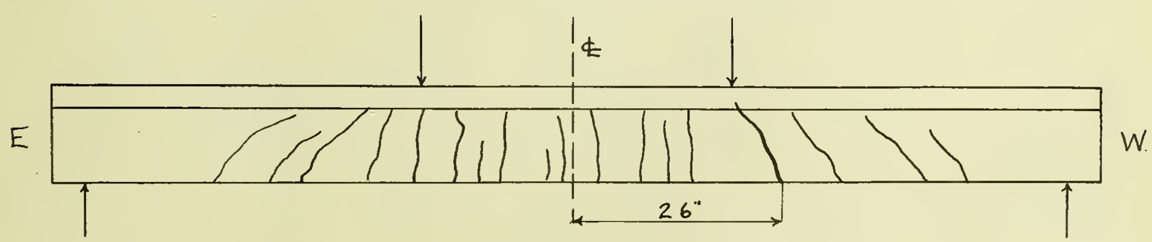
North Side.

<u>Beam No 2. Width</u> of flange 32 in. The first crack developed at 10 in. West of the center at a load of 26 000 lb. At 28 000 lb. other small tension cracks appeared at 18 in. West of the center but the next to appear was at 33000 lb. at 26 in. West of the center. These were all small tension cracks. The first diagonal crack to appear was observed on both sides at 36 000 lb. at 39 in. West of the center and reached half way to the flange. As the load increased.

a great number of new diagonal cracks appeared. At 42 000 lb. one appeared at 40 in. East and at 43 000 lb. one appeared at 48 in. East and connected with this other one. At 56 000 lb. the crack at 28 in. West reached to the flange and there was the formation of 3 or 4 small horizontal cracks at the junction of the web and flange. These were about $\frac{1}{2}$ in. long, there being no connection between them. The beam carried the load well and reached 78 300 lb. as a maximum. After the load had been gradually decreased the flange on both sides from the East end to the East load point broke off, while the West end part of the flange remained intact. Deflection at the center was .72 in. at the maximum load and .78 in. when the flange failed. The stones were broken instead of the cement failing. Failure was by diagonal tension.

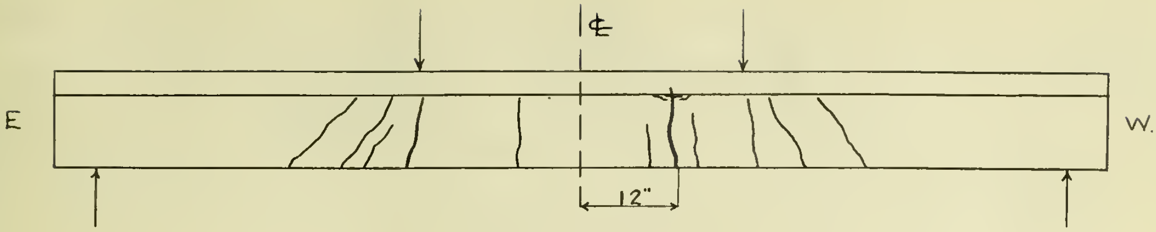


Beam No 3. Width of flange 24 in.. A small tension crack developed at 8 in West of the center at 20 000 lb. The first diagonal one appeared at a load of 28 000 lb., was 36 in. West and 2 in. high. Both tension and diagonal cracks appeared quite rapidly, especially after the 32 000 lb. load was passed. It was noticed that each diagonal crack passed through the intersection of the stirrup and the horizontal steel reinforcement. The beam carried its load well up to 52 000 lb., when it was evident that it could not stand much more. It rose, however, to 53 500 lb. At the maximum, one of the cracks at the West load point extended 1 in. into the flange. After reaching the maximum, the load was kept on and the concrete soon crushed on top under the West load point. The deflection was 1.04 in. at the maximum load and 2.22 in. when the concrete crushed. Failure was by tension.



North Side

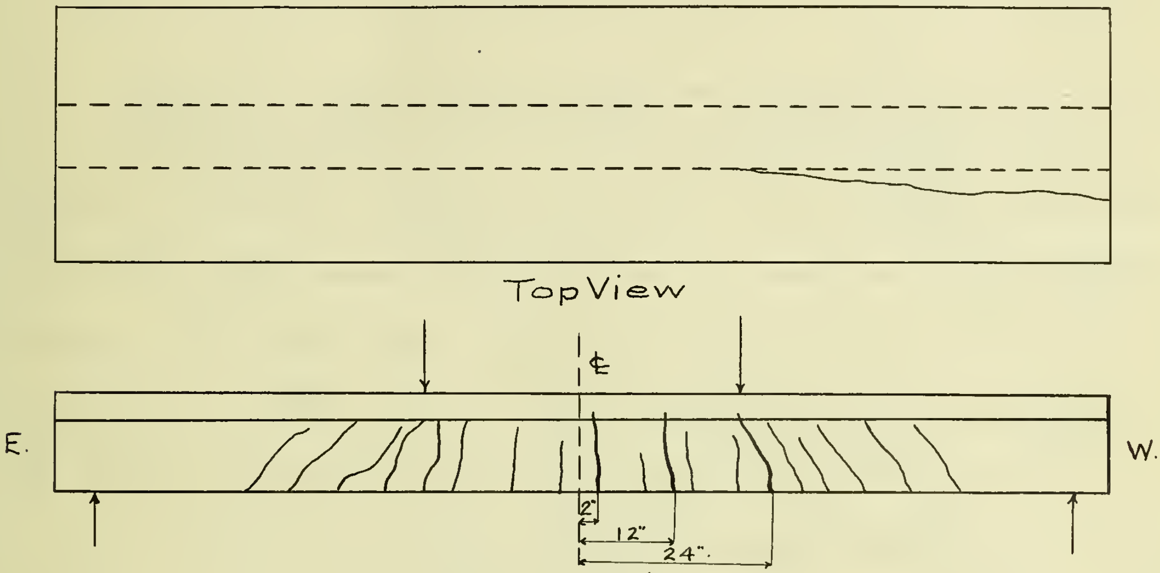
Beam No 4. Width of flange 16 in. This was the first beam on which the extensometers were used with any success. All four dials seemed to record all right up to a load of 27 600 lb., when the two South ones stopped. From that load only the North ones were used. As in the beams described above, tension cracks were the first to appear, the first one being under the West load point at a load of 18 000 lb. The first diagonal crack was noticed at 29 910 lb., was 28 in. West of the center and 4 in. high. This was the maximum load and at this point the crack which later was the principal one appeared 12 in. West of the center. After the maximum, the load fell about 2 300 lb., and then rose to 29 000 lb., after which it fell to 28 200 lb. During this interval the crack mentioned just above opened rapidly and as in Beam No 2 split along the junction of the web and flange. After rising to 29 600 lb., the load fell to 27 800 lb., rose to 28 900 lb. and at this point the concrete crushed on top at the East load point. The deflection was .30 in. at the maximum load and 1.75 in. at the point of crushing of the concrete. Failure was by tension.



North Side

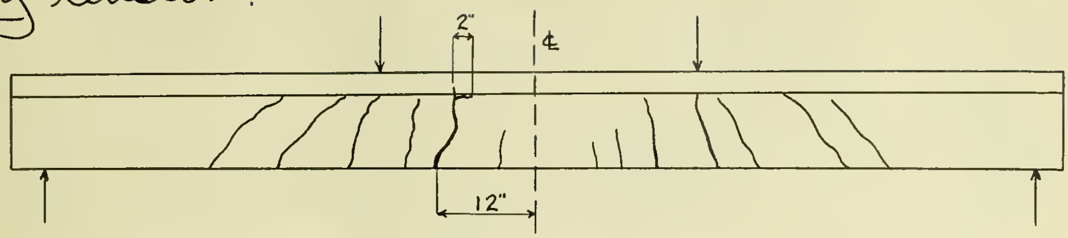
Beam No. 5. Width of flange 32 in. At a load of 25 000 lb., there was the formation of three cracks near the load points, one of these later showing up as a diagonal tension crack. The first diagonal crack to appear however was observed at a load of 35 000 lb., was 24 in. West of the center and 6 in. high. At 40 000 lb., a tension crack appeared at 12 in. West of the center and later developed into a prominent one. The last two cracks mentioned above together with a tension crack at 2 in. West, were the principal ones. At 80 000 lb. there was compression in the flange and a crack formed along the junction of the web and flange, from the West load point to the West end. The load increased very slowly. At 80 800 lb., which was the maximum observed and which was probably the maximum which the beam would have stood, one roller rolled off of its bearing and the entire beam and one pedestal fell forward, breaking the beam across at

the West load point. The deflection was 1.16 in. at crushing of the concrete and 1.40 in. at the maximum load. Failure was by tension.

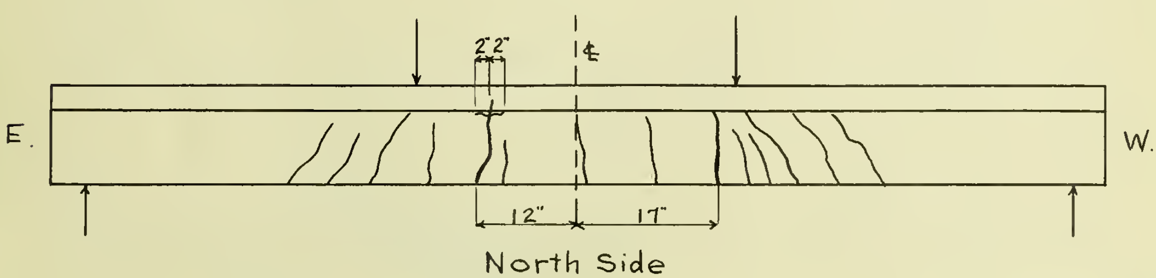


Beam No 6. Width of flange 24 in.. The first cracks appeared at 20 000 lb.. Two of these were tension and one was diagonal tension. Of the two tension cracks, the one at 12 in. East of the center later developed into the principal crack. About an equal number of small cracks developed between the load points and outside of the load points. At 37 000 lb., the maximum, the principal crack extended into the flange. When the load had fallen to 34 000 lb. it extended horizontally along at the junction of the flange and web. The deflection was .34 in. at the maximum load. Failure was

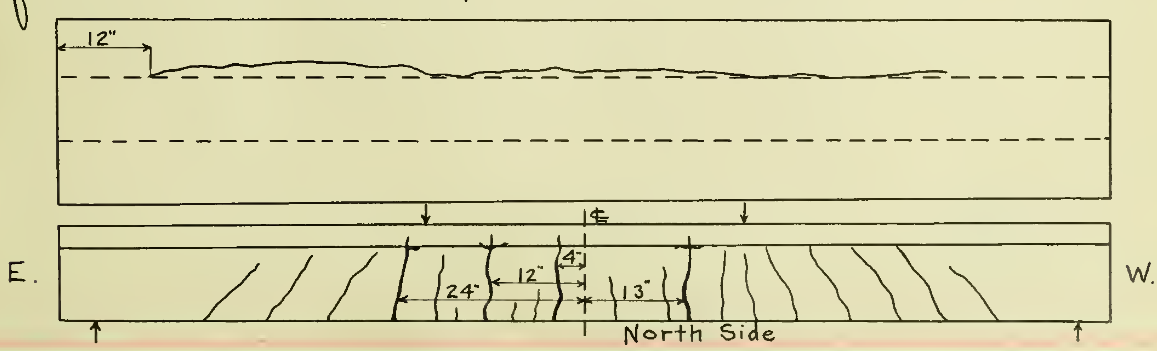
by tension.



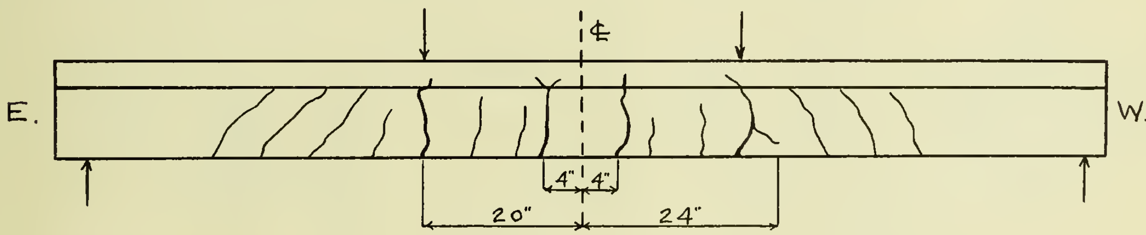
Beam No 7. Width of flange 16 in.. The first cracks appeared at 10000 lb. and as usual were small tension cracks. The first diagonal one was observed at 18000 lb. and the main crack, 12 in. East of the center, appeared at the same load. At 27000 lb. this one was $\frac{1}{16}$ in. wide and extended horizontally 2 in. on either side. At the maximum – 27300 lb. – it was $\frac{1}{8}$ in. wide and a large tension crack appeared at 1 in. West of the center. When the load had fallen to 27000 lb., a tension crack at 17 in. West was $\frac{1}{8}$ in. wide. The load gradually fell off to 26700 lb. when the concrete crushed on top at the center. The deflection was .69 in. at the maximum load and 1.93 in. at crushing of the concrete. Failure was by tension.



Beam No 8. Width of flange 24 in. This beam held its load remarkably well. Four or five times it was supposed that the maximum load had been reached but after dropping a few thousand pounds it rose again. The first tension crack appeared at 18000 lb., and the first diagonal one at 27000 lb. The first apparent maximum was 36000 lb., and at this load the principal crack was 13 in. West of the center, was $\frac{1}{32}$ in. wide and extended to the flange. At 30 000 lb. a large crack opened up at 12 in. East of the center, and also one at 24 in. East. At the final maximum – 37 300 lb. – the main cracks were as shown below, each one extending out to the edge of the flange. After reaching the maximum, the machine was run at the fast speed. With this speed the beam stood 40100 lb. but would not hold it when the machine was stopped. At 27500 lb. the concrete crushed on top and the crack shown in the top view was noted. The deflection at the maximum was 2.64 in. Failure-tension.



<u>Beam No 9</u>. Width of flange 32 in. The first tension crack appeared at 21000 lb. and the first diagonal one at 32000 lb. At 25000 lb., extensometer No. 3 refused to work and its readings were discarded during the rest of the test. The principal cracks appeared at about 40000 lb. and their positions at the maximum are shown below. After reaching the maximum – 48100 lb. – crushing soon took place on top under the East load point. The deflection was 1.57 in. at the maximum load. Failure was by tension.

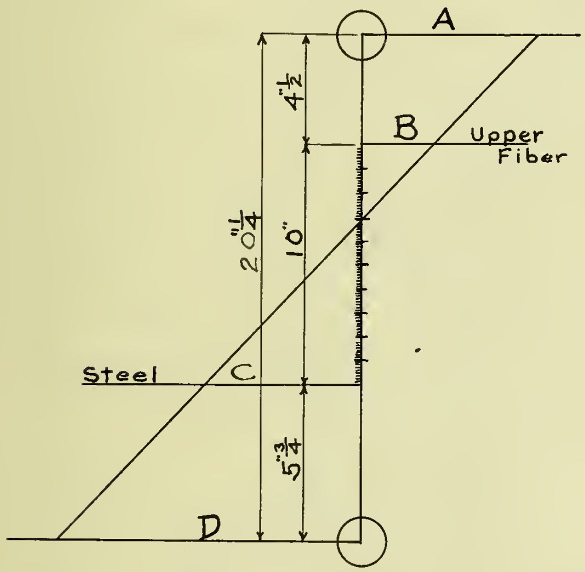


North Side

VI Discussion of Results.

The curves on pages 43, 46-48 show the unit deformations for the upper fiber and the steel and the position of the neutral axis for varying loads. The values plotted for the deformations are actual values calculated graphically from the extensometer readings. Two extensometers were used for the upper fiber and two for the steel and the average values used in each case in getting the deformations of the desired fibers.

A diagram drawn to scale on a large piece of coordinate paper was used in getting the deformations at the desired points, and the unit deformations were obtained by dividing the total deformations so found by the gage length which was 48 in.



The principle of this diagram is shown in the sketch above. A is the average readings of the upper extensometers and D of the lower. B is the total deformation of the upper fiber and C of the steel. The vertical distance from the upper fiber to the steel was divi-

ded into 100 equal parts with zero at the upper fiber. The vertical line is the plane section before bending and the inclined one the same plane after bending. The position of the neutral axis is at the intersection of these two lines. The dimensions on the sketch are the same as used in laying out the diagram.

The table on page 33 gives the stress in the steel and in the concrete at the load given in the column "Load considered". This load is somewhat below the maximum and is the last one for which the elongations and shortenings are definitely known. The column marked "K" contains values of the distance in percentages of the effective depth, of the neutral axis from the top fiber. The stress in the steel was obtained by dividing the bending moment by the product of the area of the steel reinforcement and the moment arm. The centroid of the compressive area is taken as .35 of the distance between upper fiber and neutral axis, thus giving a moment arm of the effective depth times 1 − .35 K. (See Bulletin No. 1 of the University of Illinois Engineering Experiment Station, page 21). The stress in the concrete was obtained from the formula $c = \frac{2pF}{K}$,

where c = compressive stress in pounds per square inch, p = percentage of reinforcement, f = tensile unit stress in the steel at "Load considered", and "K" the same as above.

DATA AND RESULTS.

Ref. No.	Beam No.	Flange Width in.	Amount and Kind of Reinforcement.	Area of Steel sq. in.	Per cent of Steel	Max. Load pounds
1	4	16	4 - $\frac{3}{4}$ in. Plain R'd	1.76	1.10	29 910
2	7	"	" " " "	"	"	27 300
3	8	24	5 " " "	2.20	0.92	37 300
4	9	32	7 " " "	3.08	0.97	48 100

Ref. No.	Load Considered lb.	Max. Bending Moment lb. in.	K	Moment Arm d(1-.35K)	Stress in lb. per sq. in.	
					Steel	Concrete.
1	29 910	598 000	25	9.12	37 300	3 280
2	26 200	524 000	47	8.36	35 600	1 680
3	36 000	720 000	28	9.02	36 300	2 390
4	47 300	946 000	32	8.88	34 600	2 100

From the above table it is seen that the maximum load is about reached when the steel is stressed up to its elastic limit and it seems that the load at the elastic limit of the steel may properly be taken as the ultimate strength of the beam for such a percentage of reinforcement as was used in these tests. Since the elastic limit of high steel

is greater than that of low, it is thus advantageous to use the high steel for a maximum strength. The objection to its use, that the elongation is not sufficient and that it is liable to break suddenly, does not seem to be true. In the beams that were reinforced with the high steel, there was no indication of such action. Ample warning was given long before the maximum load was reached, not only by the formation of cracks but also by the deflections, the beams with the Johnson bars deflecting almost as much as the others.

The stress in the concrete as shown in the table on the preceding page is somewhat greater than the strength developed by the cubes and cylinders as described on page 5. This is accounted for by the fact that the beams were wetted twice daily from the time of making to within a few days of being tested.

Most of the beams failed by tension in the steel long before crushing of the concrete occurred and carried loads not far from the maximum for the greater portion of the intervening time between the maximum load and crushing. During this period the neutral axis rises very rapidly as shown by the curves

of beams 7, 8, and 9 on pages 46-48. This increases the moment arms very rapidly but the stress in the steel remains practically the same as shown by computations for beams 7 and 9 in the region in which the neutral axis rises rapidly. For beam 7, the stress in the steel, as obtained from the bending moment, was 34800 lb. per sq. in. just before the maximum load and at crushing of the concrete was 36400 lb. per. sq. in.. For beam 9, the stress in the steel was 34600 lb. per sq. in. quite a while before the maximum load was reached and 33800 lb. per sq. in. a little before crushing.

From the curves of beams 4 and 7, twin beams, it is seen that the steel-deformation curve changes character at about 9500 lb. total load and at this point the steel probably took the stress that up to this time had been resisted by the tensional qualities of the concrete. It was at one time supposed that there were invisible cracks in the lower portion of the beam at this stage of the test but tests by Prof. Turneaure, who tested some moist beams upside down, showed the actual presence of such cracks. In the test of beam 7 a small crack appeared 18 in. from the center

[illegible]

at a load of 10 000 lb. The stress in the steel at the point of loss of the tensional resistance of the concrete was about 7000 lb. per sq. in. as computed by using the unit deformation and the coefficient of elasticity of the naked steel as 30 000 000.

Beams no. 2, 5, and 9 were all 32 in. wide. Of these three, no. 2 was the only one that did not have reinforcement in the flange as shown on page 14. Of these three beams, this was the only one in which the flange was sheared off. In beam no. 5 a crack formed at about the maximum load along the junction of the web and flange but the flange did not drop off on account of the transverse reinforcement. In beam no. 9 no such cracks even formed. It seems from this that the small bars imbedded in the flange are advantageous.

These tests have shown the great efficiency of the stirrups. These should extend almost to the surface of the beam so as to carry the vertical component of the diagonal tension well into the flange. They should also be placed not further apart than the effective depth of the beam on account of the

diagonal tension stresses which are a maximum along 45 degree lines when no stirrups are used. Most of the diagonal cracks in these tests were inclined less than 45 degrees from the vertical because of the stirrups. As the shear was practically uniform between support and load point, the stirrups were uniformly spaced but in structural work the spacing should be decreased as the shear increases. No trouble was experienced in slipping of the bars. One thing that helped to prevent this was the care taken in getting the concrete well placed around them and another was the reinforcement by means of stirrups. These served to transmit the longitudinal shearing stresses in the concrete to the horizontal reinforcement.

The beams developed much higher strengths than was expected. Even the smallest width of beam stood a much higher load than a rectangular beam of the same effective depth and of a width equal to the width of the web would have taken. This shows that the flanges are very effective in taking up the compressive stresses. It was expected that the larger beams would take a somewhat greater

max load

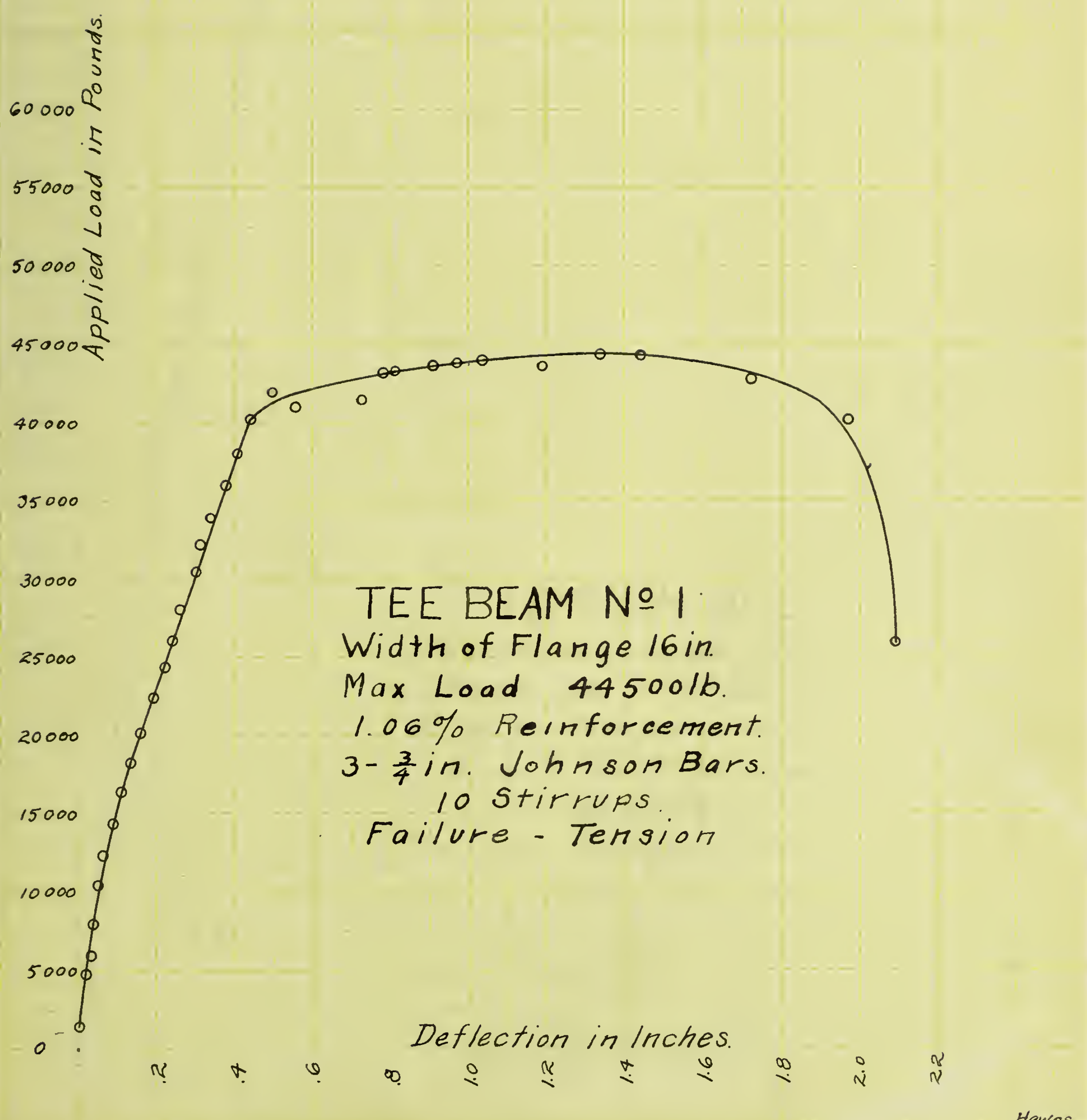
load than the smaller but the ratio was entirely unknown. For the beams with Plain

Plain Bar Beams.			Johnson Bar Beams.		
Beam No.	Flange Width. in.	~~Ultimate Str.~~ pounds.	Beam No.	Flange Width. in.	~~Ultimate Str.~~ pounds
4 + 7	16	29 000	1	16	44 000
6 + 8	24	37 000	3	24	54 000
9	32	48 000	2 + 5	32	79 000

Round steel reinforcement the 32-in. width was $1\frac{2}{3}$ times as strong and the 24-in. was about $1\frac{1}{4}$ times as strong as the 16-in. width. For the beams with Johnson bars the 32-in. one was $1\frac{3}{4}$ times and the 24-in. one $1\frac{1}{4}$ times as strong as the 16-in. beam. The strength of the 16-in. and the 24-in. beams with Plain bars and the 32-in. beam with Johnson bars is the average in each case of two tests while the other strengths are based upon only one test. These are not enough tests to furnish a sufficient number of points to plot a curve from but from the above we see that the tangent to such a curve if constructed would soon become parallel to the principal axis of the curve. When this point is reached we would have the maximum strength but the increase in strength in this region of the curve would not be in an economical ratio to the

increase in width. From the above it seems that high steel reinforcement is better than low steel reinforcement because the former causes the less decrease in strength as the width of the flange is increased. There were not enough tests, however, to establish very definite conclusions as to this point. It seems that in building construction, at least four times the width of the web would be an effective width of the flange for determining the area of the steel reinforcement in the web.

PLATE VI.



Hewes

PLATE VII

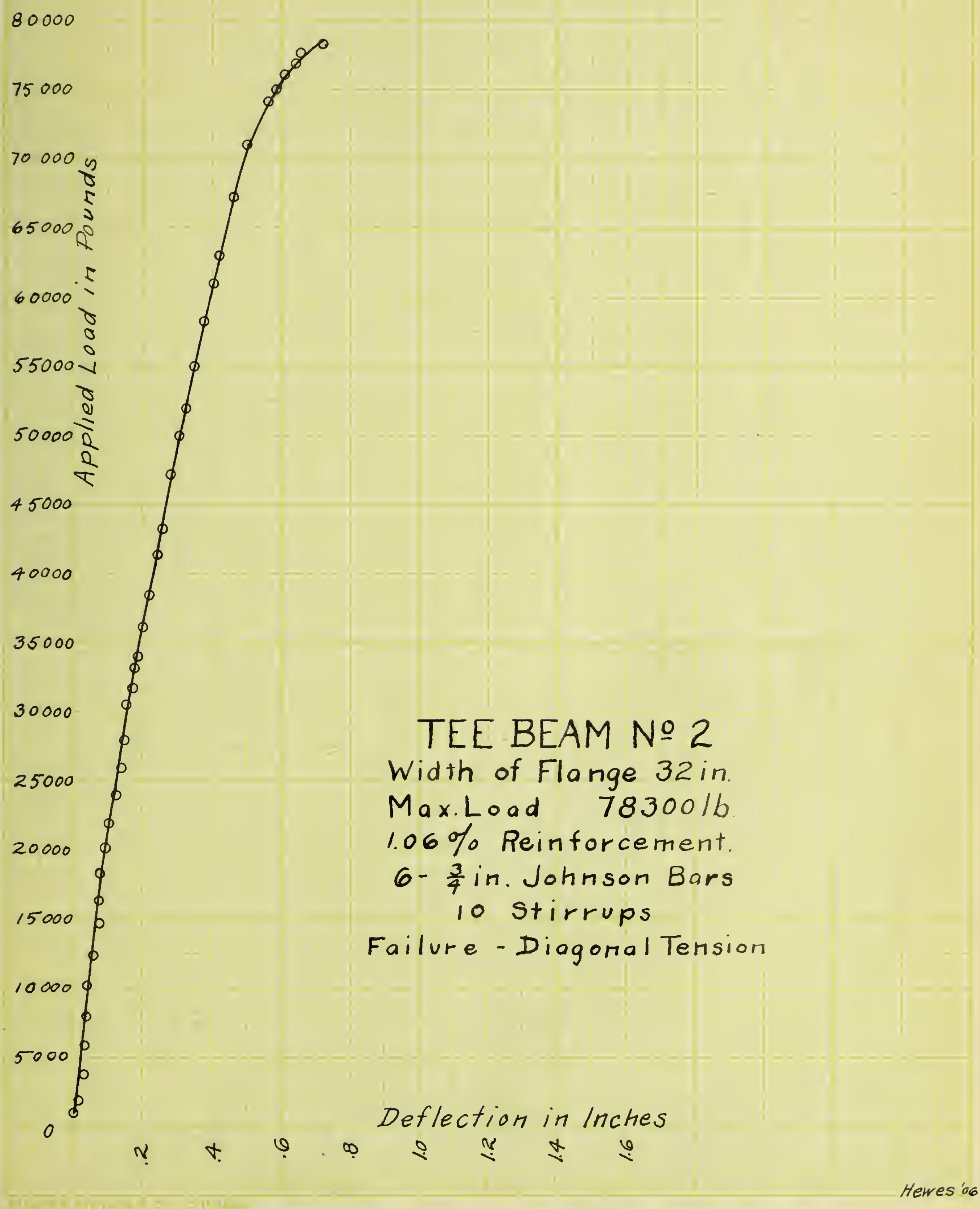
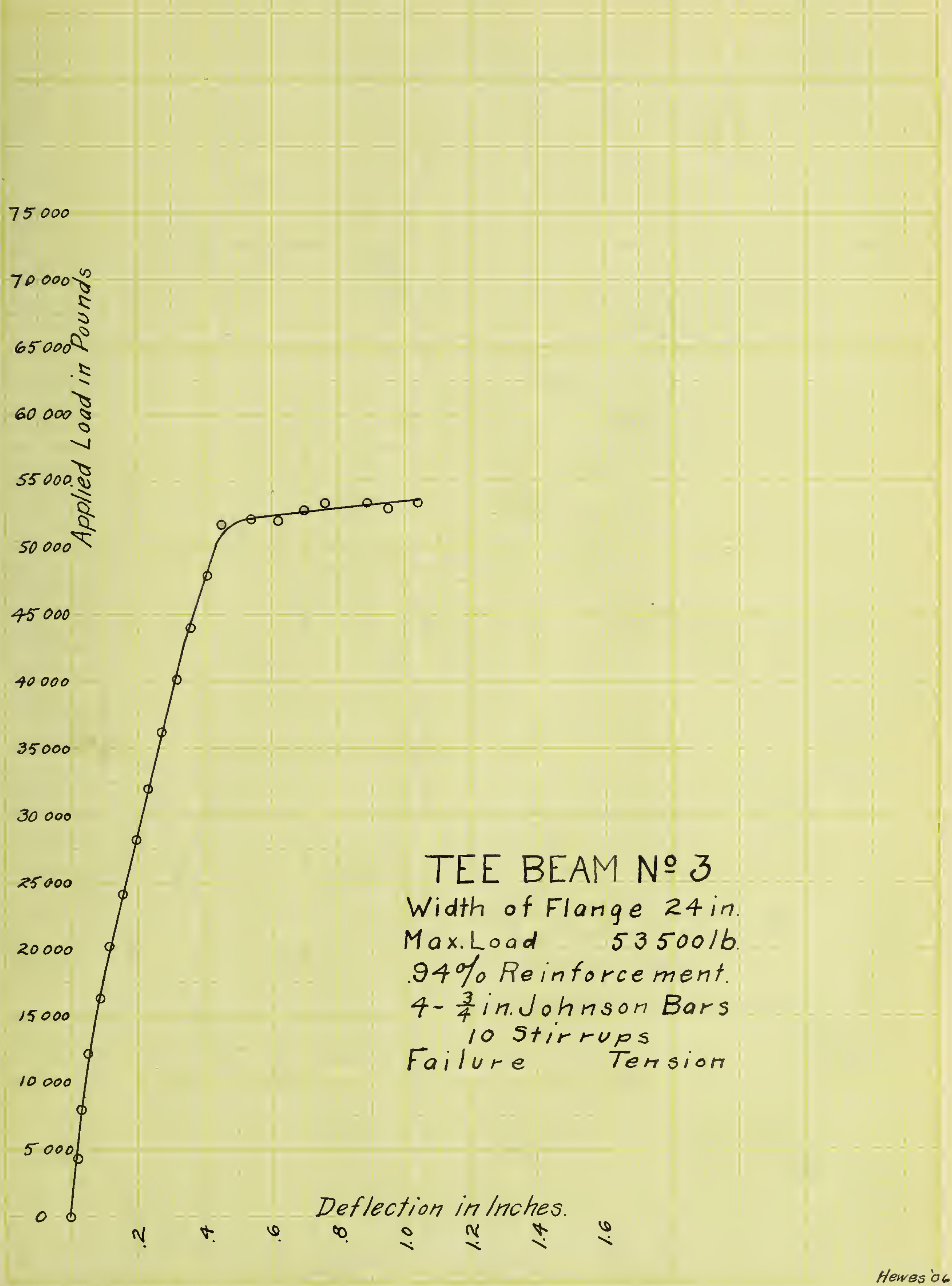

Applied Load in Pounds
80000
75 000
70 000
65000
60000
55000
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
0
Deflection in Inches
.2
.4
.6
.8
1.0
1.2
1.4
1.6
TEE BEAM Nº 2
Width of Flange 32 in.
Max. Load 78300 lb.
1.06 % Reinforcement.
6 - ¾ in. Johnson Bars
10 Stirrups
Failure - Diagonal Tension
Hewes '06

PLATE VIII.


TEE BEAM Nº 3
Width of Flange 24 in.
Max. Load 53 500 lb.
.94% Reinforcement.
4 - 3/4 in. Johnson Bars
10 Stirrups
Failure Tension
Applied Load in Pounds
0
5 000
10 000
15 000
20 000
25 000
30 000
35 000
40 000
45 000
50 000
55 000
60 000
65 000
70 000
75 000
Deflection in Inches.
.2
.4
.6
.8
1.0
1.2
1.4
1.6
Hewes '06

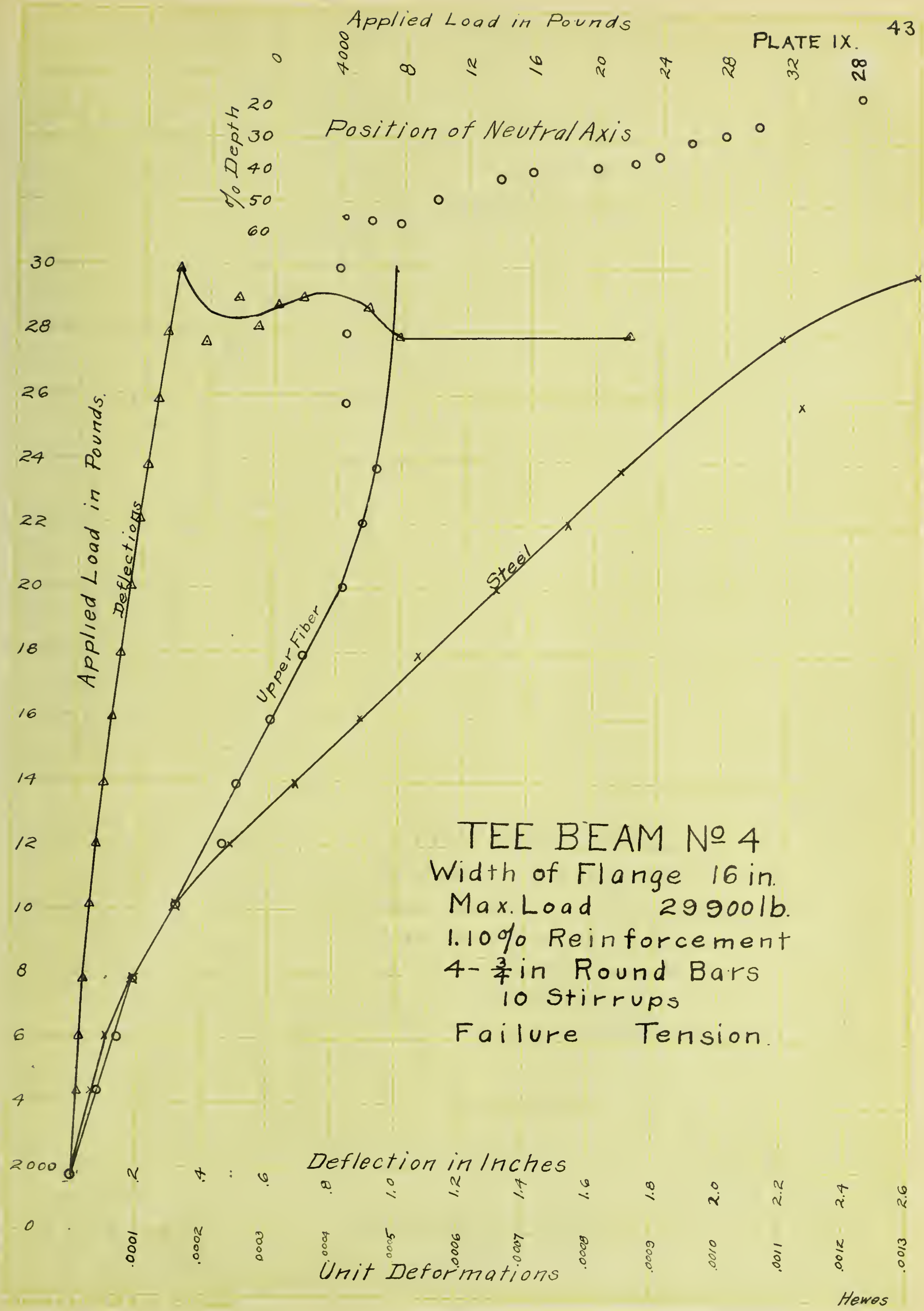
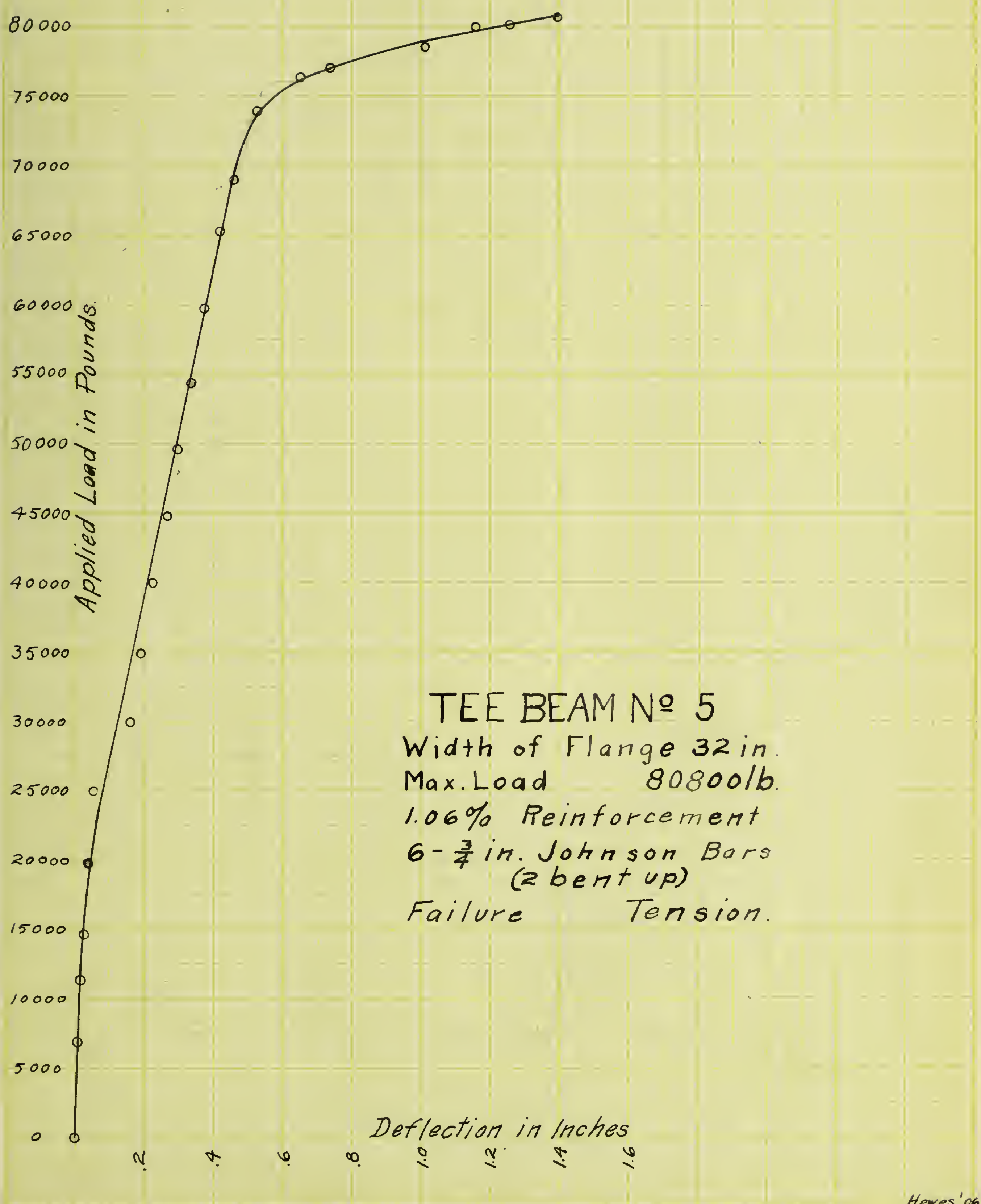

Applied Load in Pounds
0
4000
8
12
16
20
24
28
32
28
% Depth
20
30
40
50
60
Position of Neutral Axis
30
28
26
24
22
20
18
16
14
12
10
8
6
4
2000
0
Applied Load in Pounds.
Deflections
Upper Fiber
Steel
TEE BEAM No 4
Width of Flange 16 in.
Max. Load 29900 lb.
1.10% Reinforcement
4-3/4 in Round Bars
10 Stirrups
Failure Tension.
Deflection in Inches
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
.0001
.0002
.0003
.0004
.0005
.0006
.0007
.0008
.0009
.0010
.0011
.0012
.0013
Unit Deformations
Hewes

PLATE X.



80000
75000
70000
65000
60000
55000
50000
45000
40000
35000
30000
25000
20000
15000
10000
5000
0
Applied Load in Pounds.
Deflection in Inches
.2
.4
.6
.8
1.0
1.2
1.4
1.6
TEE BEAM No 5
Width of Flange 32 in.
Max. Load 80800 lb.
1.06% Reinforcement
6 - 3/4 in. Johnson Bars
(2 bent up)
Failure Tension.
Hewes '06.

PLATE XI.

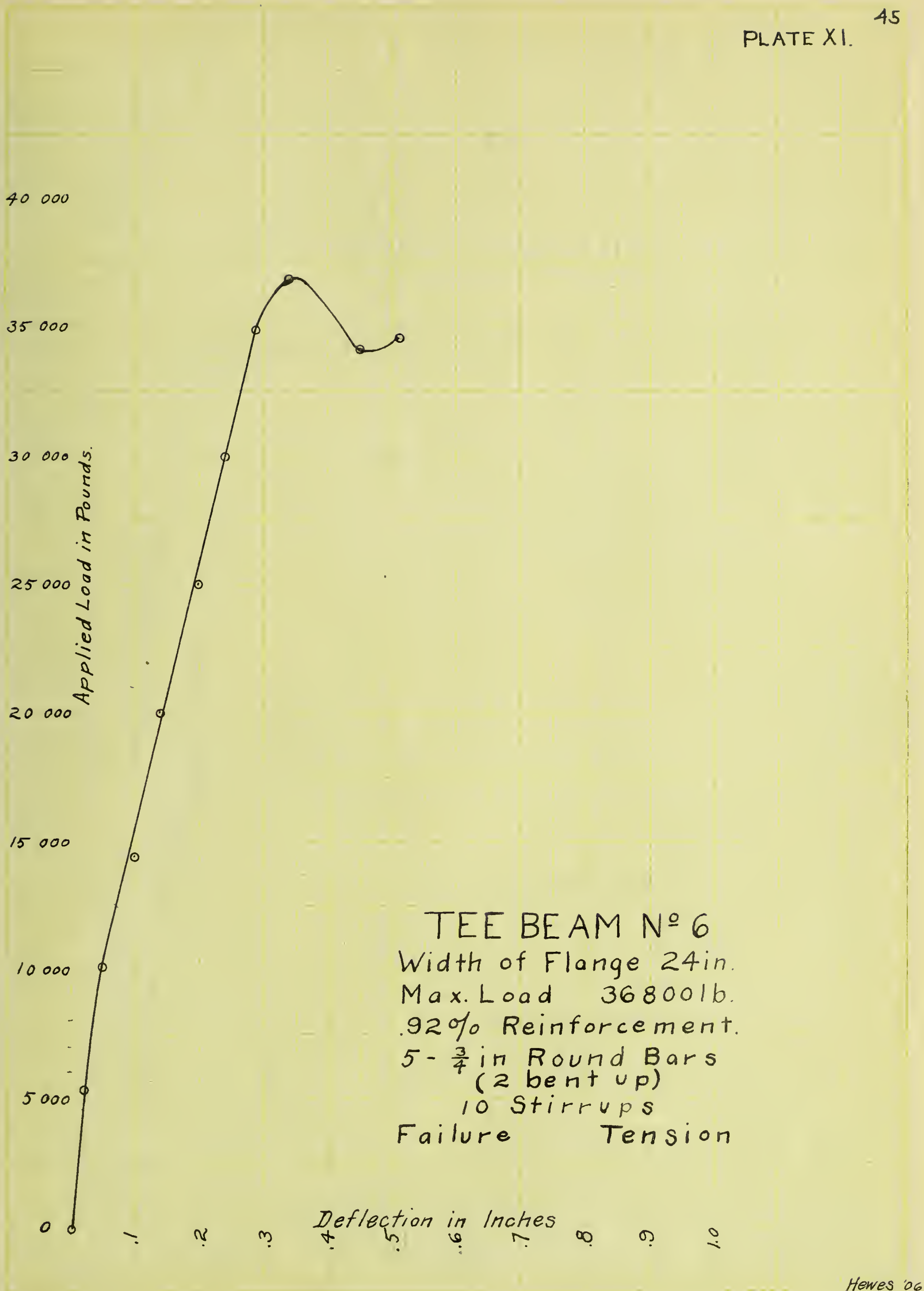
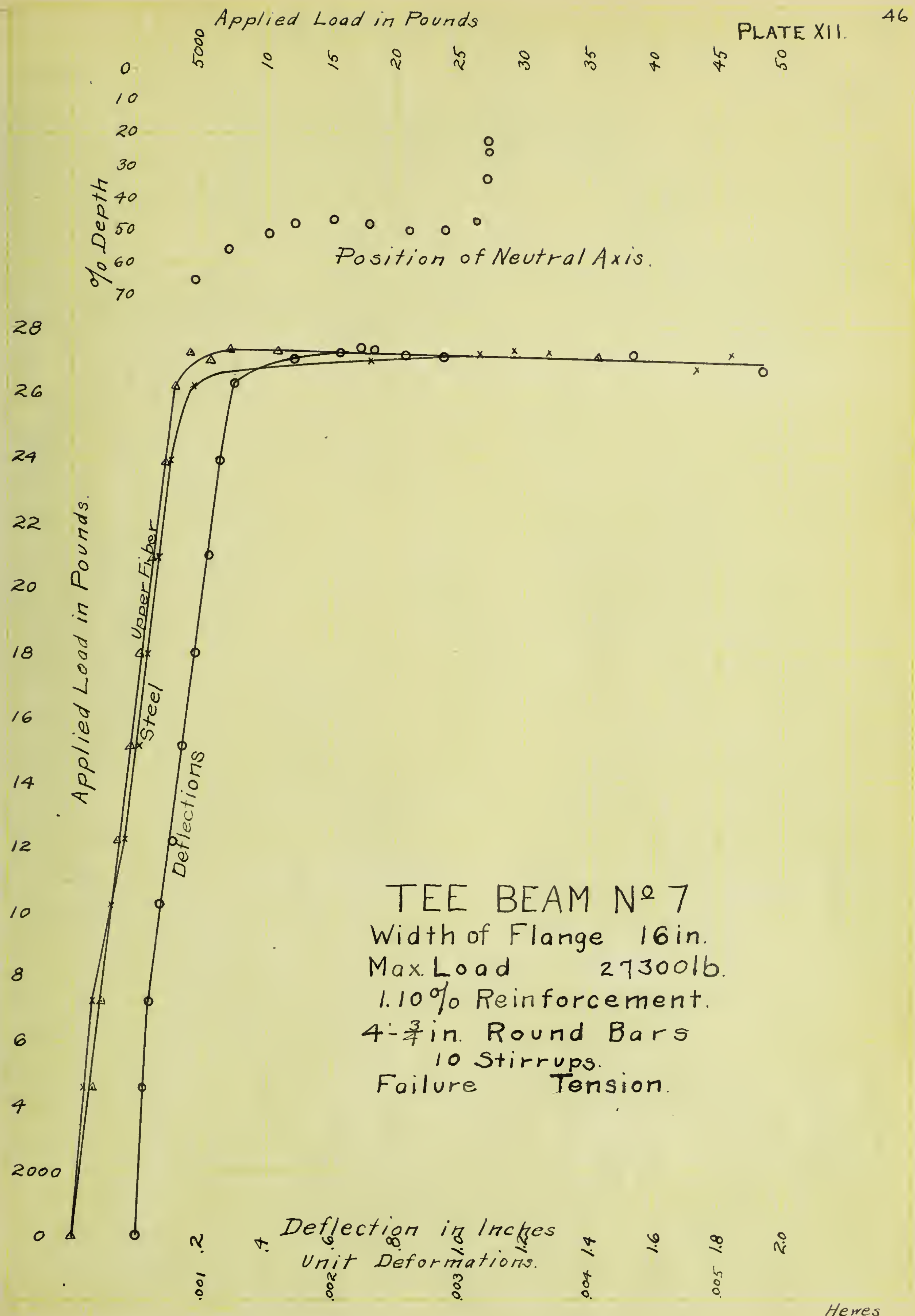
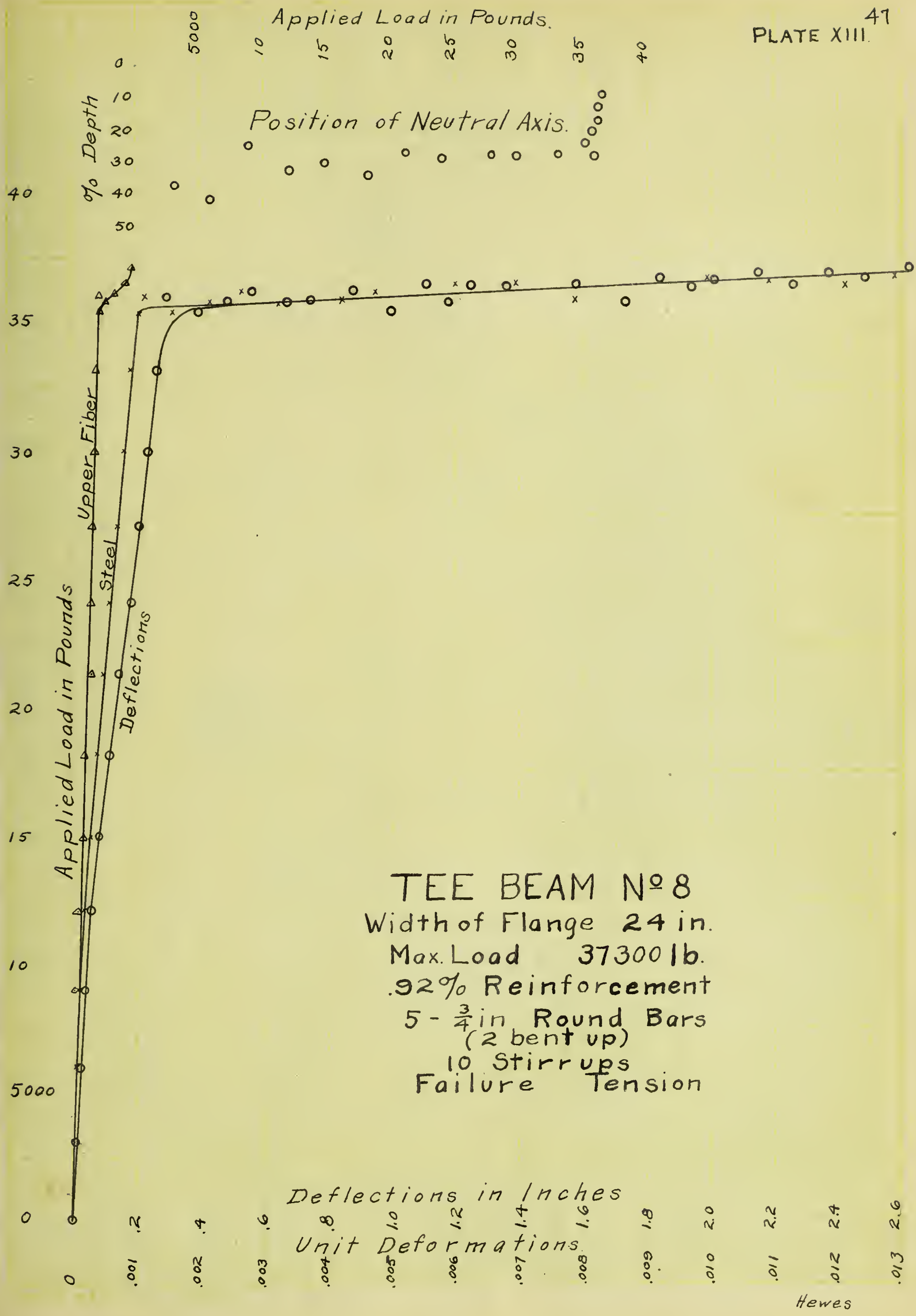

TEE BEAM Nº 6
Width of Flange 24in.
Max. Load 36800lb.
.92% Reinforcement.
5 - 3/4 in Round Bars
(2 bent up)
10 Stirrups
Failure Tension
Applied Load in Pounds.
40 000
35 000
30 000
25 000
20 000
15 000
10 000
5 000
0
Deflection in Inches
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
Hewes '06

PLATE XII.

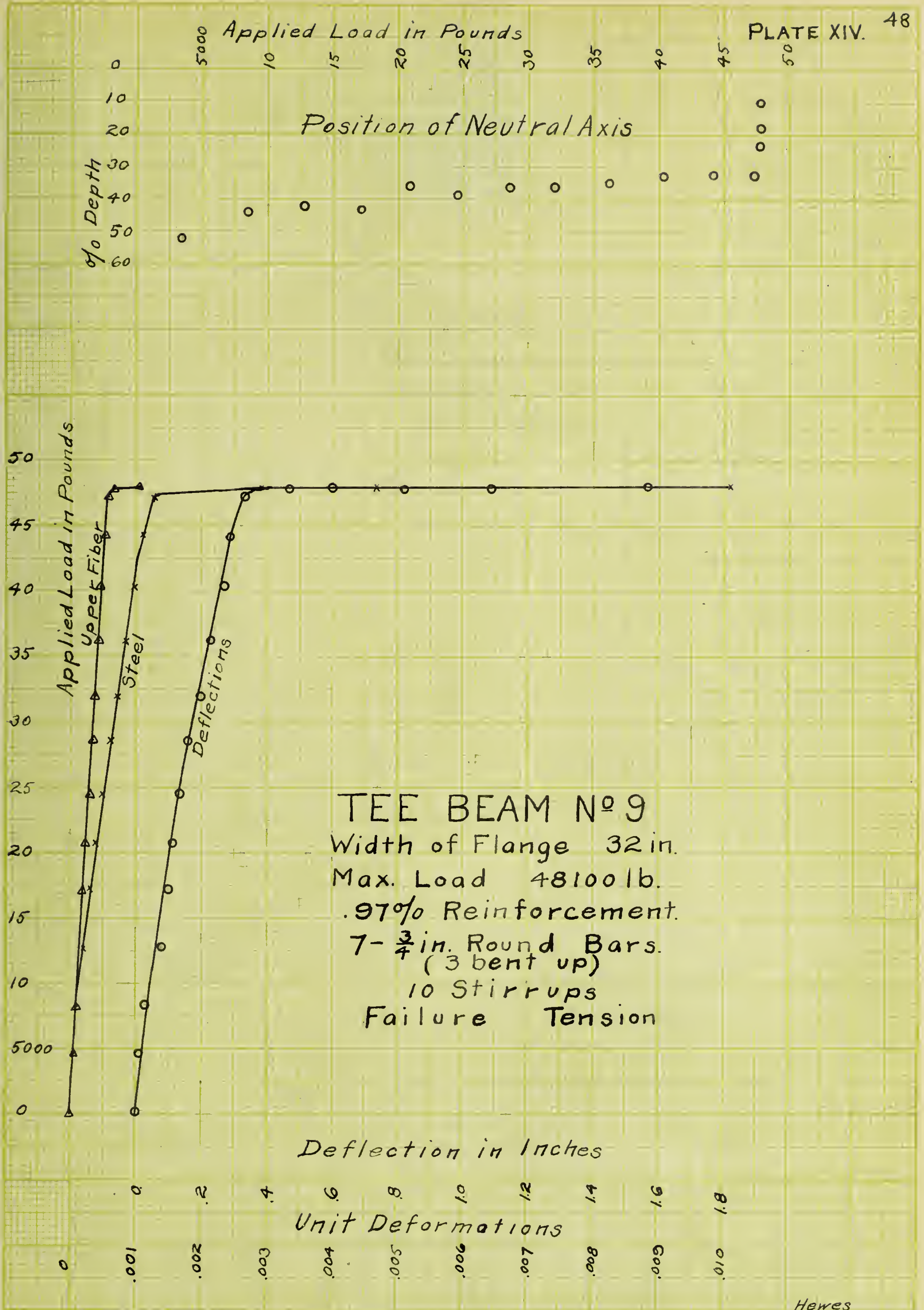


Applied Load in Pounds
0
5000
10
15
20
25
30
35
40
45
50
% Depth
10
20
30
40
50
60
70
Position of Neutral Axis.
28
26
24
22
20
18
16
14
12
10
8
6
4
2000
0
Applied Load in Pounds.
Upper Fiber
Steel
Deflections
TEE BEAM Nº 7
Width of Flange 16 in.
Max. Load 27300 lb.
1.10% Reinforcement.
4-3/4 in. Round Bars
10 Stirrups.
Failure Tension.
Deflection in Inches
Unit Deformations.
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
.001
.002
.003
.004
.005
Hewes

PLATE XIII.



Applied Load in Pounds.
5000
10
15
20
25
30
35
40
% Depth
0
10
20
30
40
50
Position of Neutral Axis.
Upper Fiber
Steel
Deflections
Applied Load in Pounds
0
5000
10
15
20
25
30
35
40
TEE BEAM Nº 8
Width of Flange 24 in.
Max. Load 37300 lb.
.92% Reinforcement
5 - ¾ in Round Bars
(2 bent up)
10 Stirrups
Failure Tension
Deflections in Inches
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
Unit Deformations
0
.001
.002
.003
.004
.005
.006
.007
.008
.009
.010
.011
.012
.013
Hewes



EUGENE DIETZGEN CO. CHICAGO

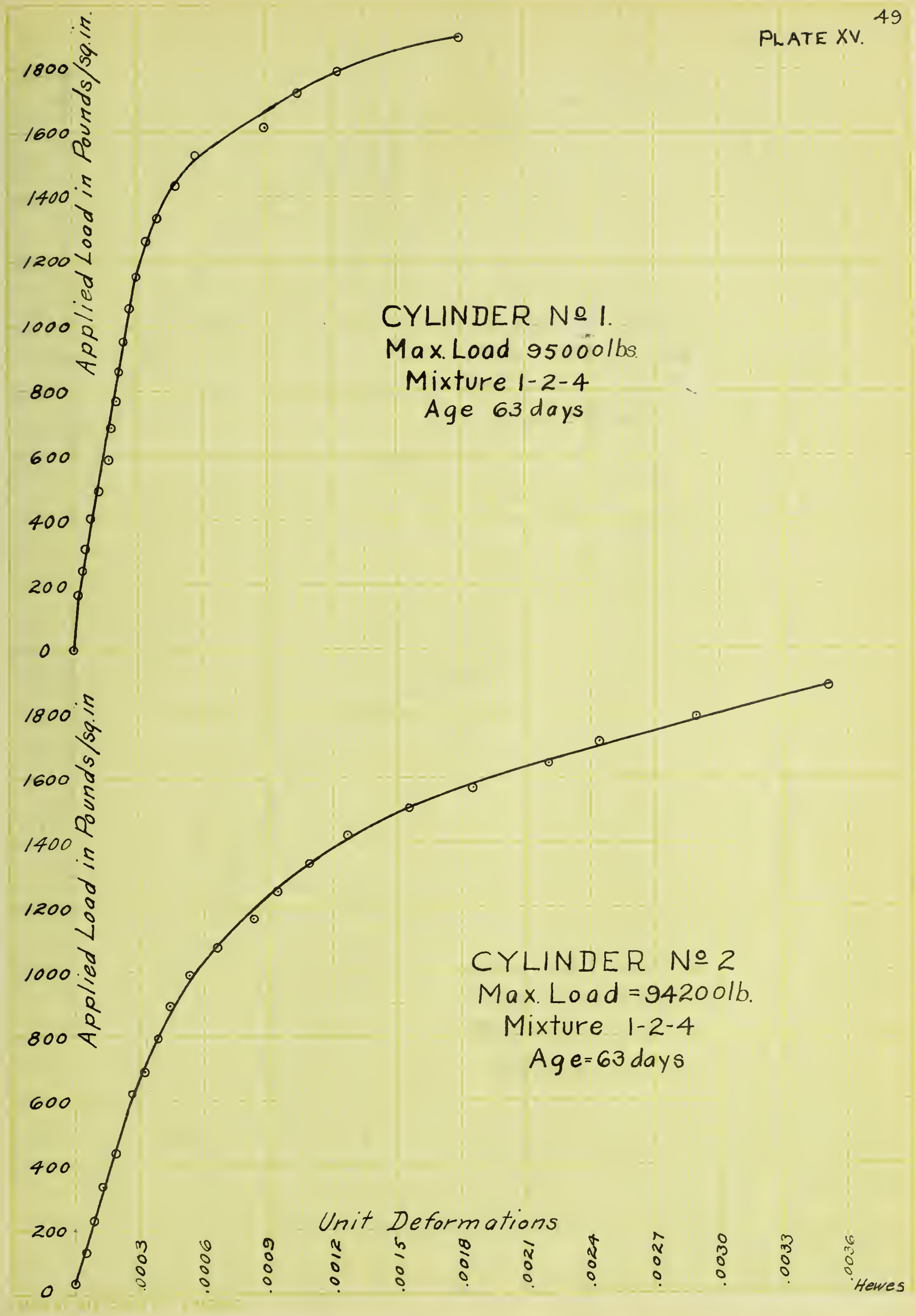
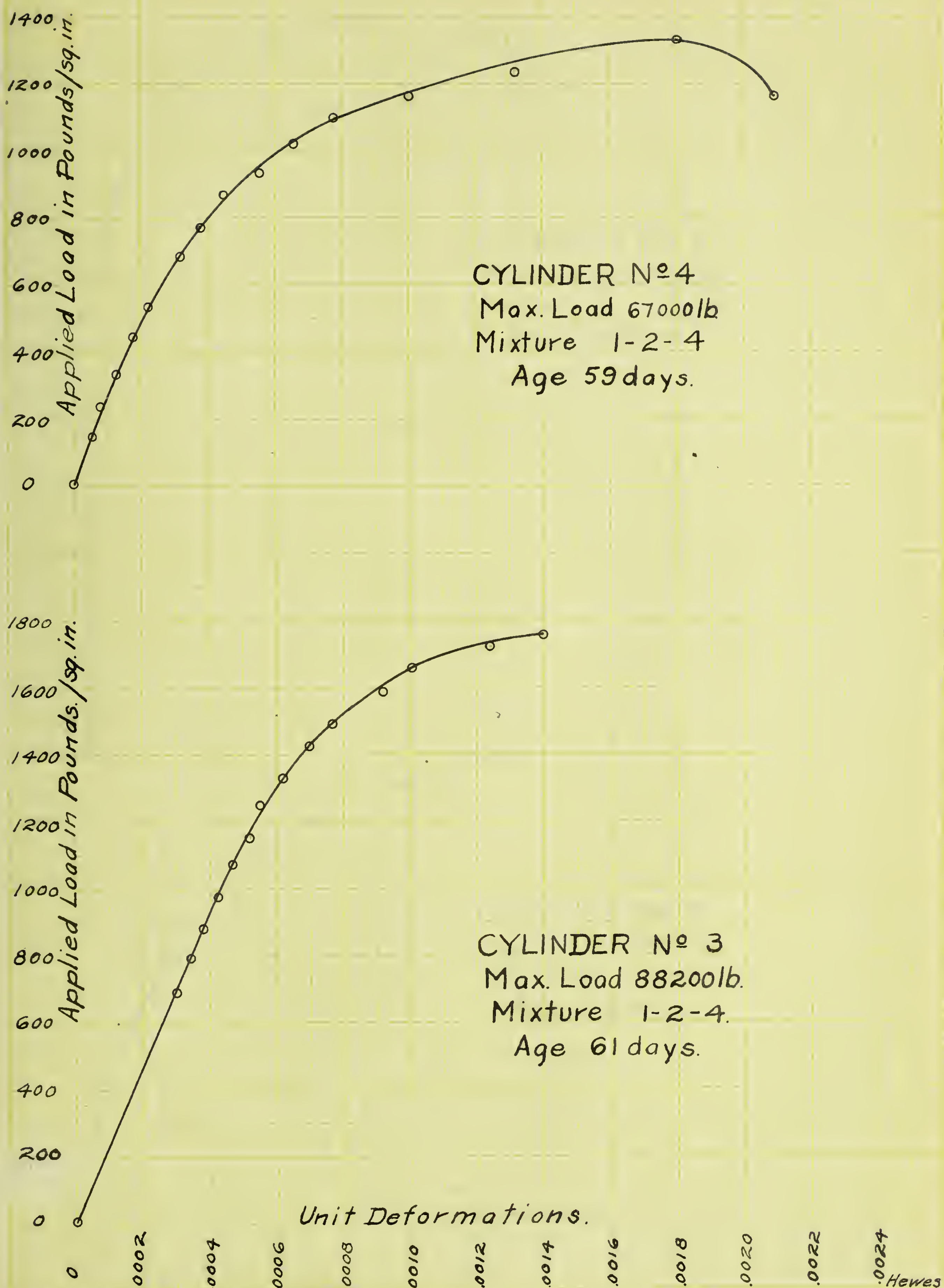


PLATE XV.
CYLINDER No 1.
Max. Load 95000 lbs.
Mixture 1-2-4
Age 63 days
CYLINDER No 2
Max. Load = 94200 lb.
Mixture 1-2-4
Age = 63 days
Applied Load in Pounds/sq.in.
Applied Load in Pounds/sq.in.
0
200
400
600
800
1000
1200
1400
1600
1800
Unit Deformations
.0003
.0006
.0009
.0012
.0015
.0018
.0021
.0024
.0027
.0030
.0033
.0036
Hewes

PLATE XVI.



Applied Load in Pounds/sq.in.
1400
1200
1000
800
600
400
200
0
CYLINDER Nº 4
Max. Load 67000lb.
Mixture 1-2-4
Age 59 days.
Applied Load in Pounds./sq.in.
1800
1600
1400
1200
1000
800
600
400
200
0
CYLINDER Nº 3
Max. Load 88200lb.
Mixture 1-2-4.
Age 61 days.
Unit Deformations.
0
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
.0024
Hewes

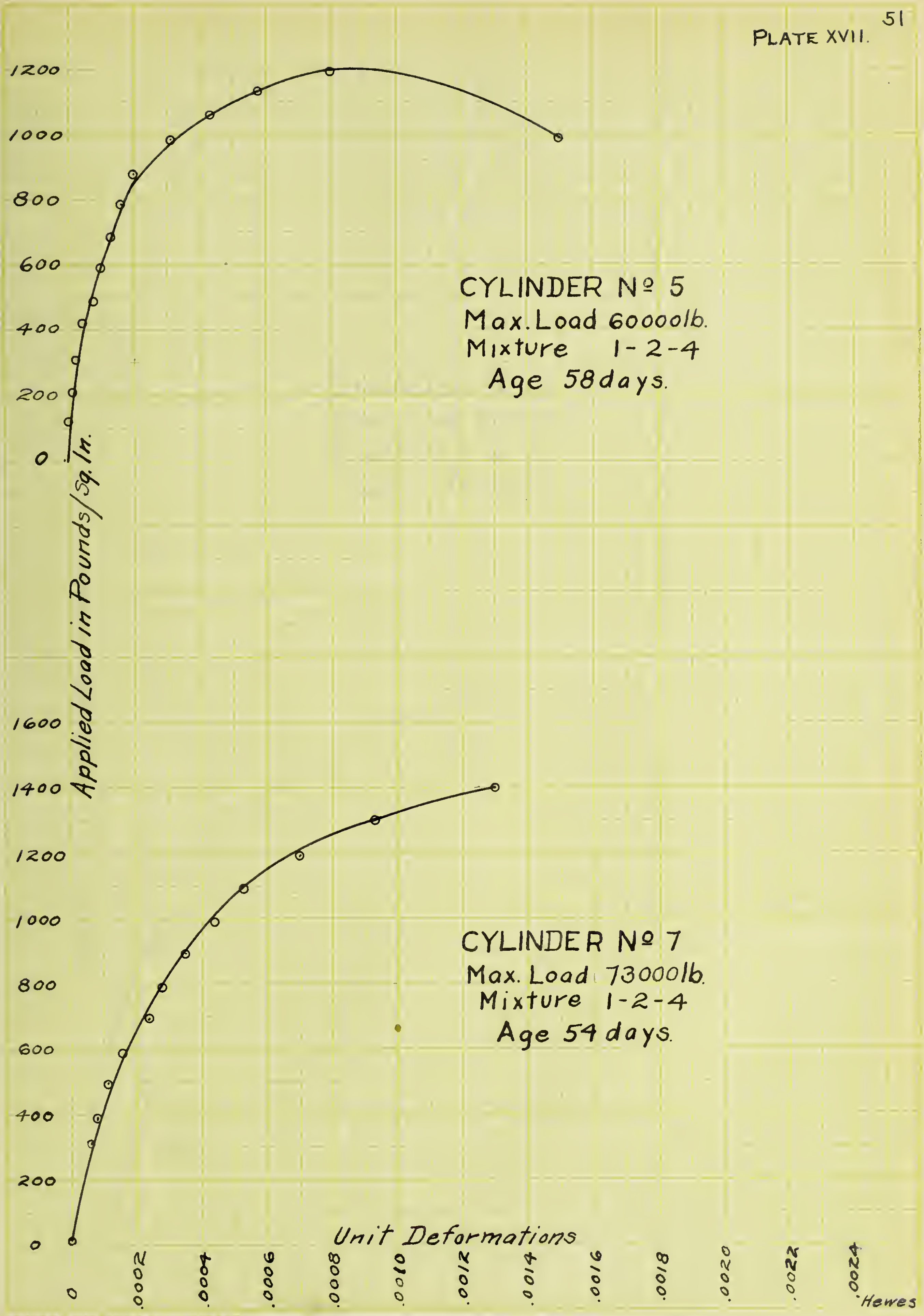
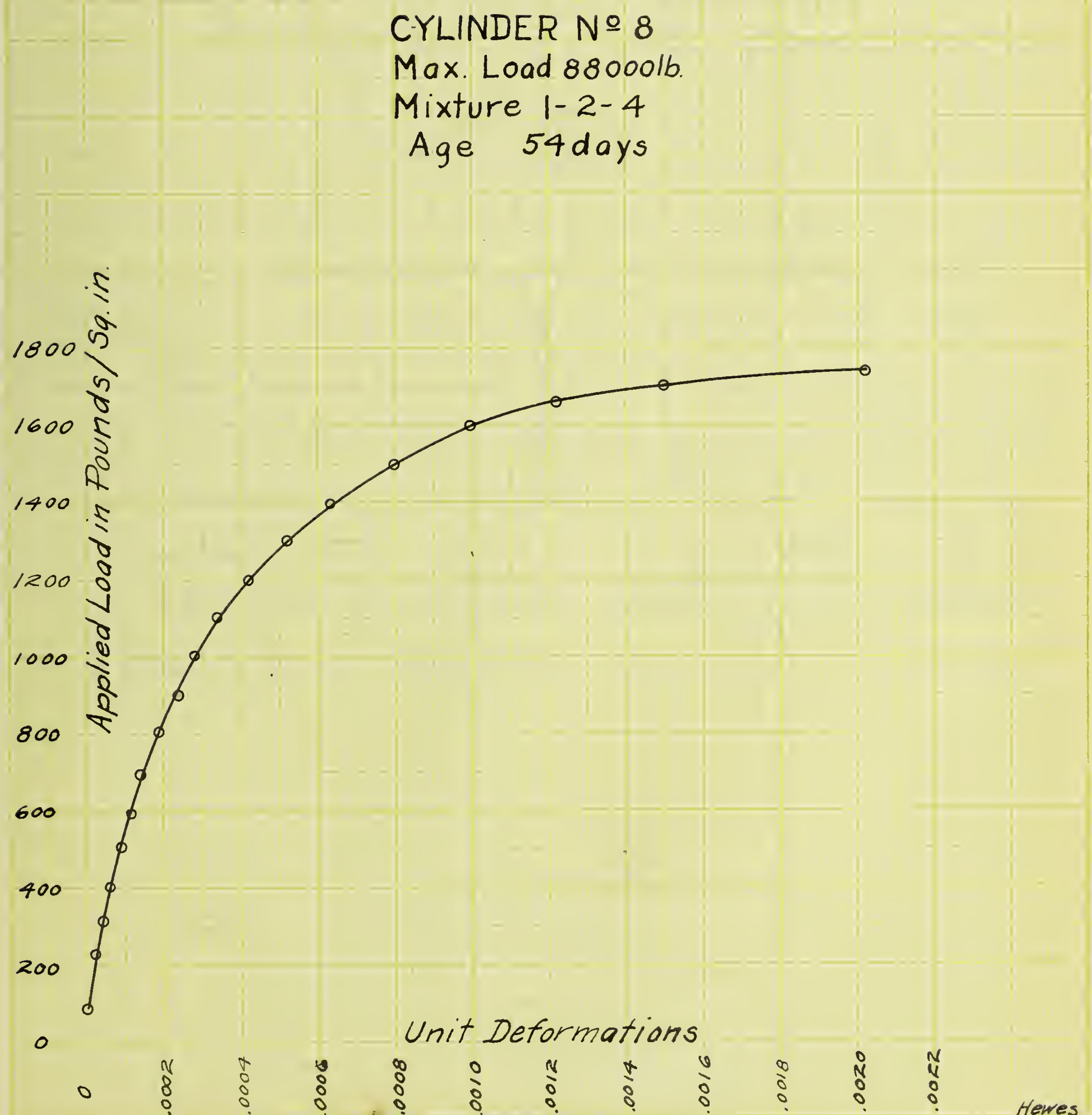


PLATE XVII.
CYLINDER Nº 5
Max. Load 60000 lb.
Mixture 1-2-4
Age 58 days.
CYLINDER Nº 7
Max. Load 73000 lb.
Mixture 1-2-4
Age 54 days.
Applied Load in Pounds/Sq. In.
Unit Deformations
0
200
400
600
800
1000
1200
1400
1600
0
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
.0024
Hewes

PLATE XVIII.


CYLINDER Nº 8
Max. Load 88000 lb.
Mixture 1-2-4
Age 54 days
Applied Load in Pounds/Sq. in.
1800
1600
1400
1200
1000
800
600
400
200
0
Unit Deformations
0
.0002
.0004
.0006
.0008
.0010
.0012
.0014
.0016
.0018
.0020
.0022
Hewes

VIII Tables of Original Readings

On the following pages are given the original readings for all the beams and a part of the derived values for those beams on which extensometer readings were taken. The column headed "Deflection" does not contain the original readings from the scale or deflectometer but instead the deflections from the position of zero load. The sub-headings, 1, 2, 3 and 4 under the heading "Extensometers" indicate the four extensometers. 1 and 3 are always the upper and 2 and 4 the lower extensometers, 1 and 4 being on one side and 2 and 3 on the other. In the two columns headed "Unit Deformations" are given the deformations per unit of length for the fiber indicated in the sub-heading, computed with reference to zero deformation at the zero of applied load, by the method explained on page 31.

Beam № 1		Beam № 2		Beam № 3	
Applied Load in lb.	Deflection inches	Applied Load in lb.	Deflection inches	Applied Load in lb.	Deflection inches
2360	.0	1160	.016	0	.0
4700	.016	2000	.030	4500	.02
6000	.030	3920	.031	7970	.03
8000	.031	5900	.034	12200	.05
10300	.046	8040	.034	16340	.09
12260	.062	10100	.060	20080	.12
14300	.090	12400	.078	24100	.16
16200	.109	14740	.078	28240	.20
18300	.130	16300	.078	32130	.23
20140	.156	18320	.078	36220	.27
22420	.187	20120	.094	40120	.32
24150	.218	22030	.100	44000	.36
26100	.245	24000	.125	48000	.41
28000	.255	26000	.141	51800	.45
30220	.295	28000	.150	52150	.54
32260	.312	30630	.156	53000	.70
34050	.343	33130	.180	53300	.76
36000	.375	35070	.190	53500	.89
38080	.406	36080	.203	53000	.95
40150	.439	38540	.219	53500	1.04
42000	.500	40300	.234		
41000	.562	42240	.250		
42460	.734	43260	.258		
43200	.781	45050	.266		
43400	.812	47200	.281		
43600	.906	49050	.297		
43800	.969	51060	.322		
44000	1.031	53020	.340		
44500	1.343	55040	.348		
44000	1.437	57290	.375		
43800	1.617	59220	.391		
42830	1.719	61000	.406		
40100	1.969	63000	.422		
26000	2.094	65280	.440		
		67170	.463		
		69000	.484		
		71000	.500		
		72020	.516		
		74200	.560		
		75000	.590		
		77000	.641		
		78300	.719		
		20000	.781		

Beam No. 4.

Appl'd Load	Original Readings					Unit Deform.		Neutral Axis
	Extensometers inches				Deflect inches	Upper Fiber	Steel	
	1	2	3	4				
1700	.0	.0	.0	.0	.0	.000	.00	0
4330	.0032	.0032	.0032	.0033	.010	.00004	.00003	55
6050	.0060	.0059	.0054	.0059	.020	.00007	.00005	56
7800	.0091	.0098	.0083	.0091	.030	.00009	.00009	57
10100	.0150	.0175	.0135	.0162	.050	.00016	.00016	49
12000	.0205	.0255	.0217	.0238	.069	.00023	.00024	49
13950	.0260	.0339	.0234	.0315	.089	.00025	.00034	42
16000	.0321	.0433	.0290	.0403	.111	.00030	.00044	40
17900	.0379	.0524	.0341	.0485	.136	.00035	.00053	40
20040	.0449	.0640	.0403	.0579	.163	.00041	.00065	39
22300	.0520	.0760	.0430	.0642	.190	.00044	.00076	37
23830	.0540	.0826	.0460	.0696	.210	.00046	.00084	35
25800	.0540	.0922	.0460	.0764	.238	.00041	.00112	30
27870	.0548	.1032	.0510	.0851	.269	.00041	.00109	28
29910	.0551	.1090	.0520	.0940	.305	.00040	.00130	25
27600	.0552	.1152	.0520	.1362	.390	.00028	.00155	16

Beam No. 5		Beam No. 6	
Applied Load in lb.	Deflection inches	Applied Load in lb.	Deflection inches
0	.0	0	.0
7000	.010	5470	.020
11270	.020	10150	.050
14560	.030	14500	.100
19660	.040	20060	.140
25000	.060	25080	.200
29930	.170	30000	.240
34900	.200	34930	.290
40000	.230	36800	.340
44800	.270	34140	.450
49600	.300	34600	.510
54500	.340		
59700	.380		
65240	.420		
69000	.470		
74000	.530		
76300	.650		
77000	.740		
78500	1.010		
80000	1.160		
80140	1.260		
80800	1.400		

Beam № 7

Appl'd Load	Original Readings					Unit Deform.		Neutral Axis
	Extensometers inches				Deflect. inches	Upper Fiber	Steel	
	1	2	3	4				
0	.0	.0	.0	.0	.0	.00	.00	0
475	.0004	.0007	.0005	.0100	.0			
4550	.0129	.0101	.0123	.0110	.02	.00016	.00001	65
7230	.0212	.0189	.0158	.0197	.04	.00022	.00017	56
10200	.0309	.0313	.0236	.0302	.07	.00030	.00029	51
12200	.0382	.0410	.0288	.0392	.11	.00036	.00040	48
15100	.0489	.0570	.0373	.0475	.14	.00046	.00052	47
18000	.0550	.0683	.0438	.0478	.18	.00053	.00057	48
21000	.0637	.0823	.0521	.0515	.22	.00063	.00065	50
23850	.0722	.0960	.0594	.0550	.25	.00072	.00075	50
26200	.0826	.1155	.0696	.0700	.30	.00080	.00093	47
27000	.1250	.2170	.1144	.1635	.49	.00106	.00230	34
27200	.1470	.2910	.1147	.2360	.63	.00092	.00315	22
27300	.1560	.3220	.1610	.2640	.69	.00121	.00341	26
27200	.1660	.3440	.1720	.2880	.73	.00125	.00369	25
27100	.1890	.3950	.1953	.3400	.83	.00140	.00433	24
27100	.2150	.4500	.2220	.3970	.95	.00158	.00509	24
26700	.2830	.4720	.4670	.4050	1.93	.00407	.00431	48

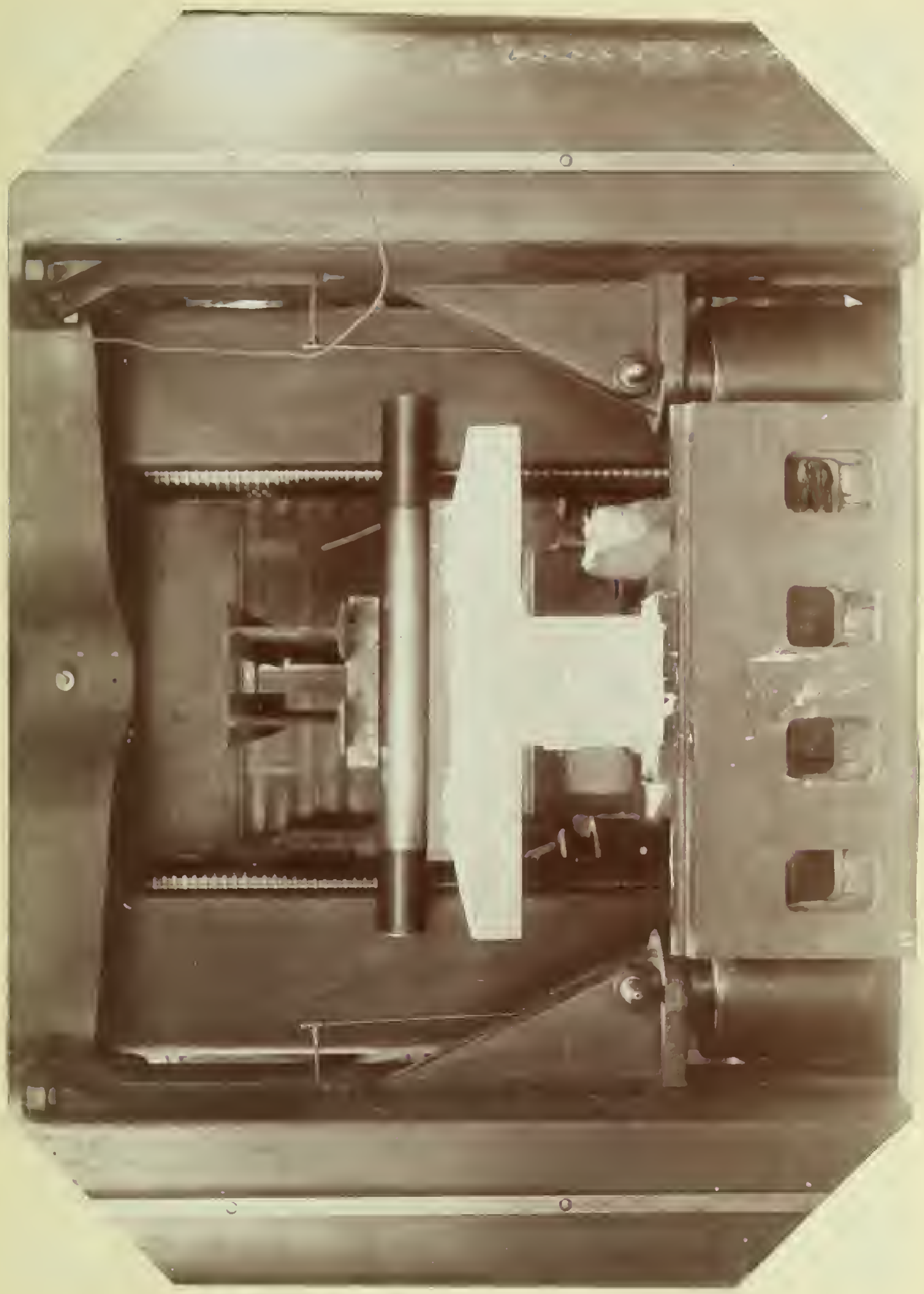
Beam № 8

Appl'd Load	Original Readings					Unit Deform.		Neutral Axis
	Extensometers inches				Deflect inches	Upper Fiber	Steel	
	1	2	3	4				
0	.0	.0	.0	.0	.0	.00000	.00000	
3190	.0015	.0020	.0022	.0039	.01	.00002	.00003	38
6000	.0046	.0059	.0057	.0080	.02	.00005	.00007	42
9050	.0046	.0137	.0102	.0160	.04	.00010	.00020	25
12170	.0151	.0258	.0155	.0248	.06	.00020	.00030	33
15000	.0217	.0382	.0200	.0330	.09	.00020	.00040	30
18240	.0278	.0493	.0232	.0407	.12	.00030	.00050	34
21270	.0330	.0606	.0283	.0501	.15	.00029	.00060	27
24140	.0376	.0695	.0333	.0562	.19	.00032	.00072	29
27200	.0421	.0785	.0385	.0652	.21	.00036	.00083	28
30000	.0458	.0862	.0443	.0740	.24	.00040	.00093	28
33100	.0507	.0964	.0501	.0840	.27	.00044	.00105	27
36000	.0547	.1043	.0550	.0920	.30	.00047	.00113	28
35400	.0652	.1480	.0680	.1195	.40	.00047	.00159	23
35800	.0830	.2020	.0853	.1605	.49	.00053	.00219	20
36200	.0915	.2390	.0980	.1901	.57	.00054	.00265	17
35700	.1010	.2870	.1140	.2310	.67	.00055	.00323	15
35800	.1220	.3730	.1340	.2818	.75	.00056	.00421	12
36200	.1446	.4130	.1568	.3410	.88	.00069	.00474	13
36500	.1615	.5080	.1888	.4310	1.11	.00057	.00600	9
36800	.2070	.8270	.2920	.7300	1.73		.01010	
37000	.2420	1.0401	.3540	.8840	2.15	.00052	.01210	4
37300	.3440	1.2460	.4552	1.1410	2.64	.00094	.01540	6

Beam № 9

Appl'd Load	Original Readings					Unit Deform		Neutral Axis
	Extensometers inches				Deflect inches	Upper Fiber	Steel	
	1	2	3	4				
0	.0	.0	.0	.0	.0	.0	.0	
4750	.0043	.0055	.0042	.0039	.01	.00005	.00004	52
8400	.0087	.0110	.0084	.0092	.03	.00009	.00010	44
12680	.0149	.0196	.0145	.0190	.08	.00015	.00020	42
17150	.0225	.0310	.0220	.0274	.10	.00022	.00030	43
20800	.0268	.0402	.0230	.0354	.11	.00023	.00040	36
24630	.0328	.0505	Extensometer not Working.	.0430	.13	.00031	.00050	39
28600	.0373	.0600		.0530	.16	.00034	.00062	36
32000	.0424	.0692		.0606	.20	.00039	.00071	36
36200	.0485	.0815		.0690	.23	.00044	.00083	35
40400	.0539	.0960		.0781	.27	.00047	.00098	32
44170	.0598	.1090		.0857	.29	.00052	.00110	32
47300	.0670	.1240		.0955	.33	.00058	.00124	32
47850	.0940	.2030		.1641	.47	.00067	.00219	23
47900	.1240	.2640		.2250	.60	.00087	.00292	23
47900	.1729	.3850		.3776	.82	.00104	.00467	18
47900	.1900	.5410		.4720	1.09	.00079	.00644	10
48100	.2210	.8170		.7090	1.57	.00004	.01009	1

PLATE XVIII.



BEAM No. 2 IN PLACE
AFTER TESTING.
WEST END.

PLATE XIX.

BEAM No. 2 IN PLACE
AFTER TESTING
SHOWING SHEARING OFF OF FLANGE.
EAST END.

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